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BUREAU OF WATER AND WASTEWATER
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Baseline Analysis and Capacity Assessment Report
Project 1028

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Baltimore High Level Sewershed
Baseline and Capacity Analysis
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EXECUTIVE SUMMARY

This Report defines the performance of the High Level Collection System in its Baseline configuration and under Baseline evaluation conditions, described below. The Report evaluates the system capacity under a series of wet weather conditions, and identifies the system elements that limit system capacity.

The Baseline condition for each element of the Collection System is defined by:

- System pipe sizes and shapes, and O&M conditions as they existed at the time of inspection and flow monitoring (2007);
- Completion of all Paragraph 8 projects;
- All known City of Baltimore Capital Improvement Projects (CIP) that would affect the capacity of the system and are scheduled to be completed by 2016; and
- Wet weather RDII projected under 7 different storm intensities with return frequencies from 3 months to 20 years.

The Future condition includes all of the Baseline definitions as well as the following modifications:

- Domestic, commercial and industrial base sanitary flow (BSF) quantities predicted for 2025; and
- Base infiltration increased to 110% of 2007 quantities to account for pipe deterioration.

This report, therefore, represents the system performance at the current or Baseline conditions as well as the Future condition in 2025 if no action is taken to increase capacity or remediate continuing condition deterioration.

The Baseline dry weather analysis shows the system has adequate capacity under dry weather conditions. Some surcharges are projected under dry weather conditions; the most significant are due to discharge from Ashburton Wash Water Lake. None of the projected surcharges would cause dry weather SSOs.

The Baseline wet weather analysis shows the system will not overflow at any location in the three-month storm. The system will overflow 3 million gallons in a 1-year storm, and 6.2 million gallons in a 2-year storm. The SSO volume exceeds 20 million gallons in a 20-year storm.

The High Level Collection System has over 7,000 feet of pipe with inadequate size to accommodate the 3-month design storm without surcharging. In a 10-year storm, 28,000 feet of pipe are of inadequate size. The extent of inadequate-size pipe increases to 55,000 feet with a 20-year storm.

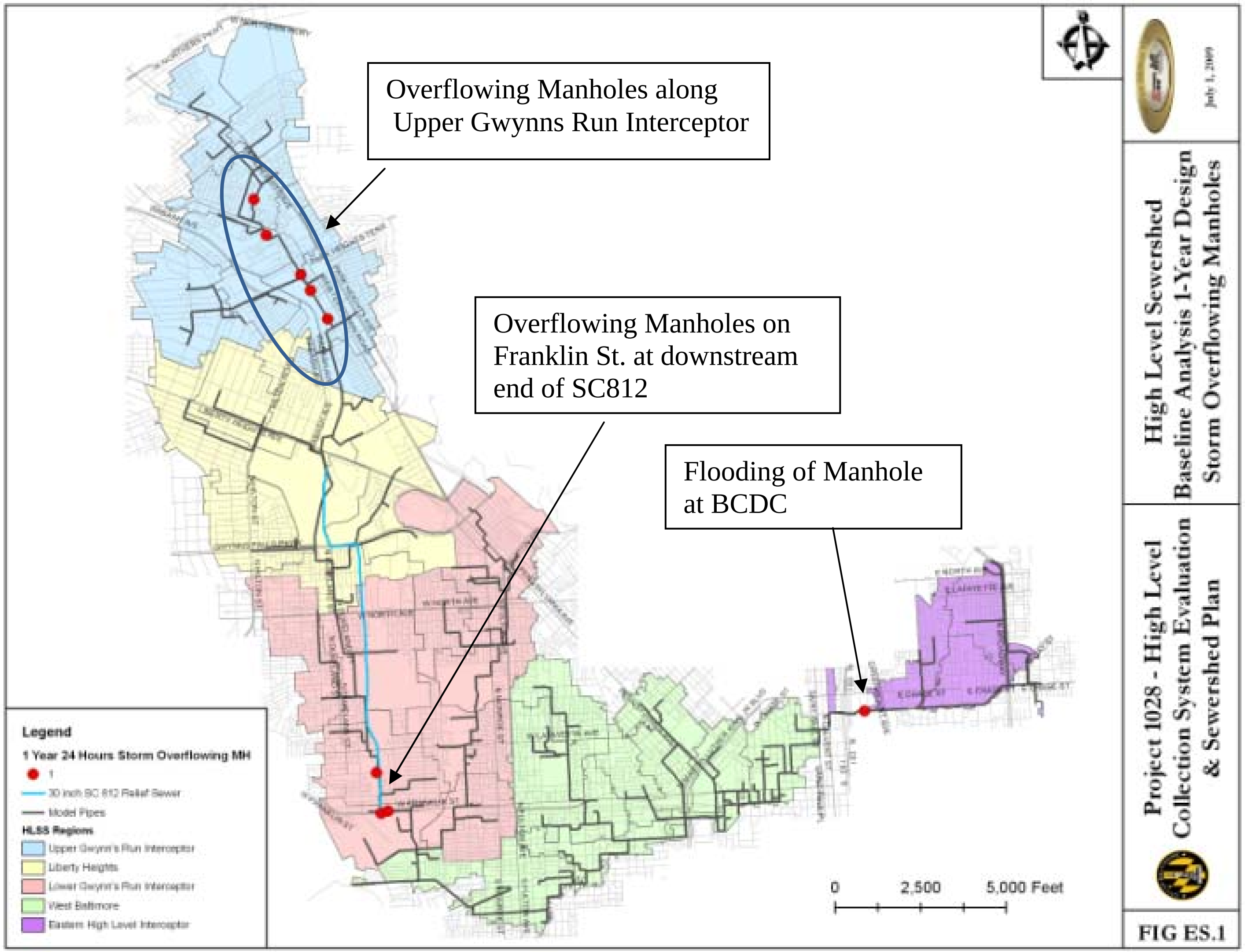


Figure ES.1: High Level Sewershed Baseline 1-Year Design Storm Flooding of Manholes

SECTION 1

PROJECT DESCRIPTION

1.1 STUDY AREA

The High Level Sewershed (HLSS) has a drainage area of approximately 4,600 acres (7.2 square miles) served by separate storm and sanitary sewers. The majority of HLSS drainage area is residential, with a total population of about 100,000 based on the 2000 U.S. Census data. This drainage area generally slopes in the north-south direction with higher ground elevations in the Northern portions.

A schematic diagram of HLSS with its major elements and boundary conditions is shown in Figure 1.1. Wastewater from Northwest portion of the HLSS drainage area is collected by the Gwynn's Run Interceptor (GRI), which in turn, joins the larger High Level Interceptor (HLI) at the south end of the GRI. The HLI runs from west to east receiving flow contributions from the HLSS in the upstream reach, and from the Jones Falls and Low Level Sewersheds in the downstream reaches. The HLI joins the Outfall Interceptor at the beginning of Outfall Sewershed, and the Outfall Interceptor eventually conveys flow to the Back River Wastewater Treatment Plant (WWTP) for treatment. There are no permanent pump stations within the HLSS service area.

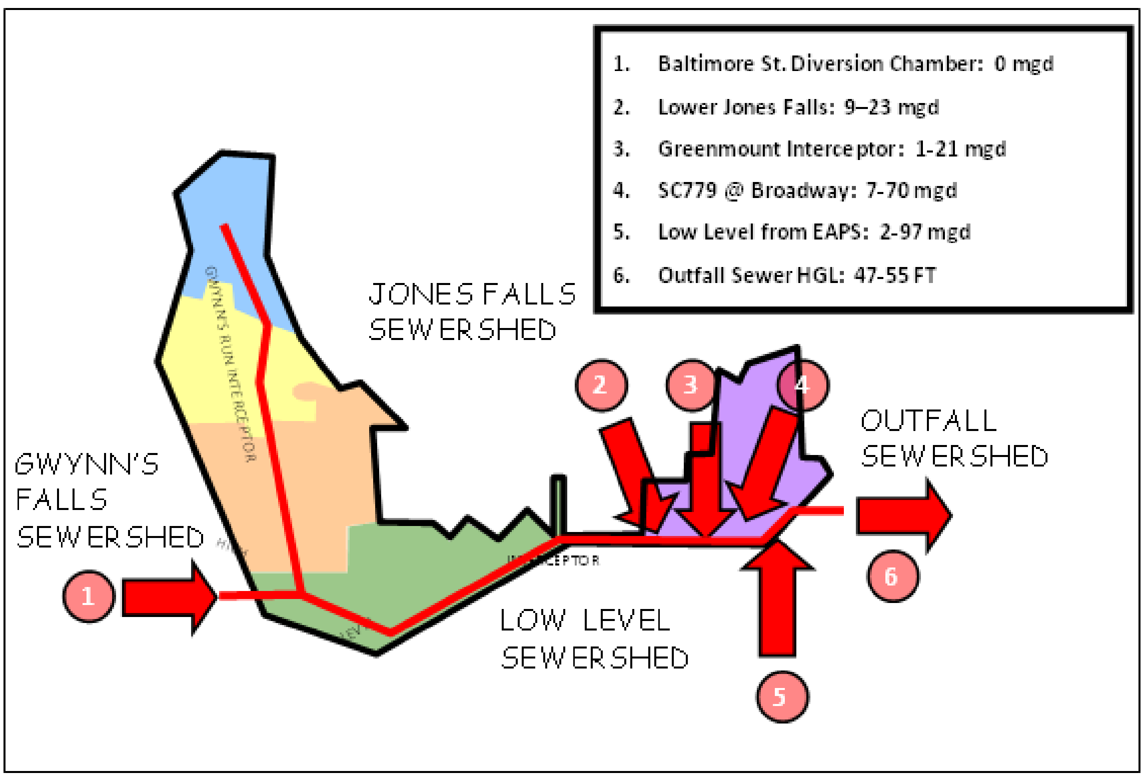


Figure 1.1: High Level Sewershed with Major Elements and Boundary Conditions

The HLSS has been subdivided into five different sub-sewersheds listed in Table 1.1. The boundaries for each of the sub-sewersheds are depicted on Figure 1.2.

Table 1.1: Sub-Sewersheds Within the High Level Sewershed

Upper Gwynn's Run Interceptor
Liberty Heights
Lower Gwynn's Run Interceptor
West Baltimore
Eastern High Level Interceptor

1.2 CONSENT DECREE REQUIREMENTS

For systems with no permanent pumping stations, as stipulated in Section 12.B of the CD, it is required that the hydraulic model must be capable of determining:

1. Volume of wastewater flow in Major Gravity Lines;
2. Hydraulic pressure or hydraulic grade line ("HGL") of wastewater at any point in the Major Gravity Lines; and
3. Likelihood and location of overflows(s) within a service area under high flow conditions.

Paragraph 9.C of the CD requires the City to determine the range of storm events for which the collection system in its existing condition can convey peak flows without the occurrence of sanitary sewer overflows (SSOs). As part of the analyses, the model must identify and include all components of the collection system that cause or contribute to flow restrictions or that have the potential to cause or contribute to overflows.

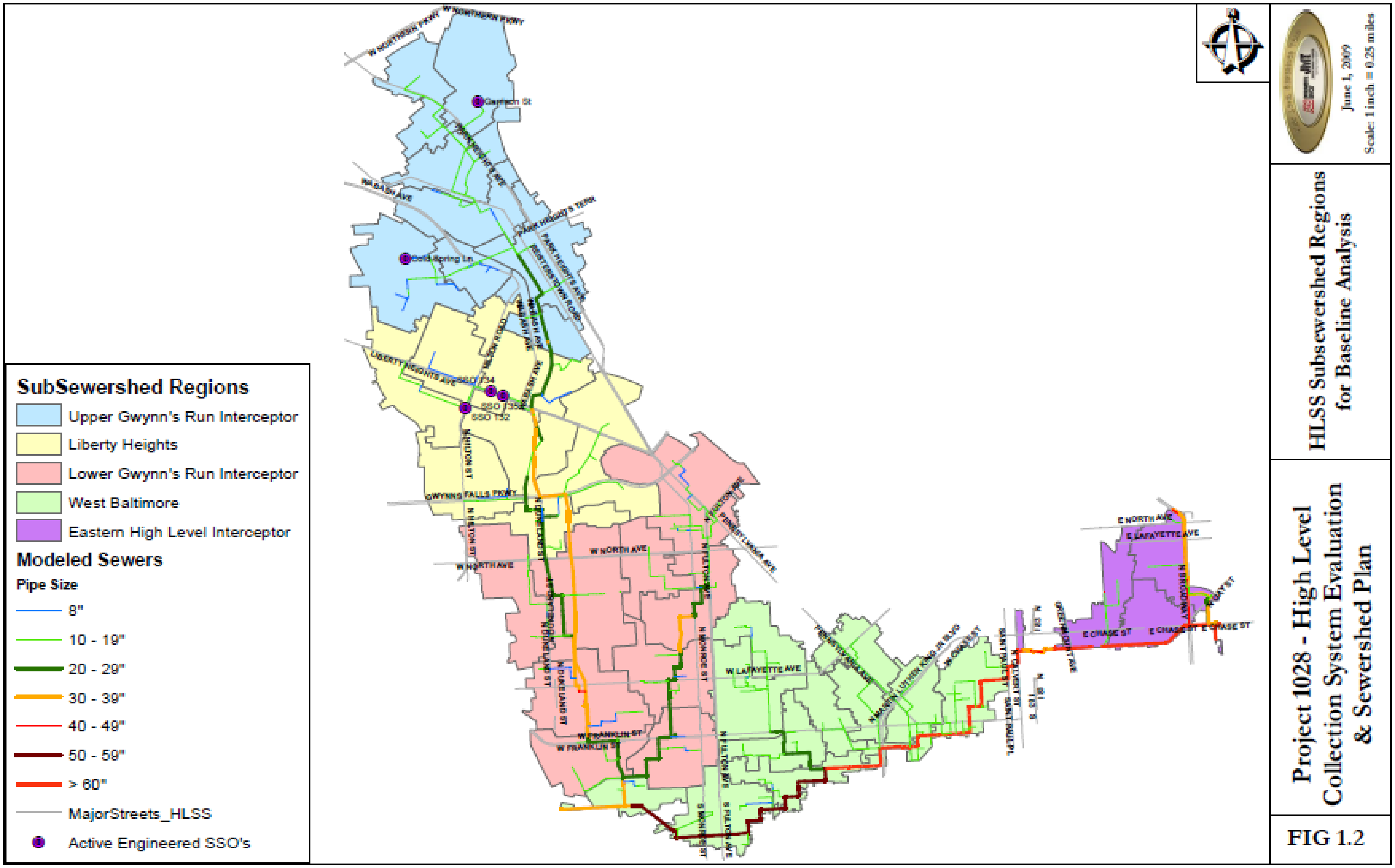


FIGURE 1.2: HLSS SUBSEWERSHED REGIONS FOR BASELINE ANALYSIS

1.3 GUIDELINES AND REQUIREMENTS

Important guidelines for this Baseline Analysis and Capacity Assessment are found in the City's BaSES Manual. The CD requires that the model be used to analyze the collection system under baseline and future conditions for seven design storms. These design storms include: a three-month storm having a duration equal to the time of concentration for the sewershed (2.5 hours); a 20-year 24-hour duration storm and 1-, 2-, 5-, 10-, and 15-, and 20- year 24 hour storms.

The Baseline condition is defined as the conditions effective at the time of flow monitoring. Additionally, all Paragraph 8 projects as well as any proposed Capital Improvement Projects (CIP) scheduled for completion before Year 2016 are to be included in the Baseline conditions model. As of now, there are no CIP improvements to the HLSS planned and scheduled for completion by Year 2016. The details on Paragraph 8 projects in the HLSS are provided in the following section.

Future conditions are defined as the sewer system configuration forecasted for the Year 2025. Future conditions include Year 2025 base sanitary flows (BSF) for the projected population and employment changes. The hydraulic impact of pipe deterioration by the year 2025 is accounted for by increasing the groundwater infiltration by 10 percent over the planning period.

Modeled sewers in Figure 1.1 are categorized based on width for the interceptor sewers and diameter for all other most sewers.

SECTION 2

HYDRAULIC MODEL

2.1 HYDRAULIC MODEL DEVELOPMENT

The City of Baltimore evaluated the various modeling software currently available and selected InfoWorks CS, by Wallingford Software, Ltd, as the modeling software for the City's collection system evaluation and sewershed planning studies.

The CD requires that the City selected model network will include all force mains, major gravity lines, and pumping stations and their respective related appurtenances. The major gravity lines are defined in the CD as:

- all gravity lines ten inches in diameter or larger;
- all eight-inch gravity lines that convey or are necessary to accurately represent flow attributable to a service area in each of the Collection System's sewershed service areas;
- all gravity lines that convey wastewater from one pumping station service area to another pumping station service area; and
- all gravity lines that have caused or contributed to, or that the City knows are likely to cause or contribute to, capacity-related overflows.

The HLSS model includes all manholes, junctions, and structures along the modeled sewer lines and all control structures (e.g. sluice gates and pumping stations) existing within the system, as needed to accurately portray the collection system performance. The developed HLSS model includes over 200,000 linear feet of pipes and approximately 1,000 manholes. The model uses a horizontal datum of Maryland State Plane Coordinate System NAD83 and a vertical datum of NAVD88.

The High Level Team used the City's wastewater geodatabase to create a model network database of pipes and nodes to be used in the hydraulic model. This network database is called the "physical model". The portion of the entire HL collection system included in the physical model is called the "modeled extent". Field data obtained through land surveys and manhole inspections were used to create a model network database of the physical model. JMT is continuing to update the HLSS GIS database as final field are obtained. The HLSS team will conduct synchronization process between the GIS database and hydraulic model when the GIS database process is completed.

The HLSS receives significant inflow contributions from other sewersheds as shown in figure 2.0. Their rates are significant under both dry and wet weather conditions. Table 2.1 summaries the dry and wet weather flow rates from all the HLSS boundaries.

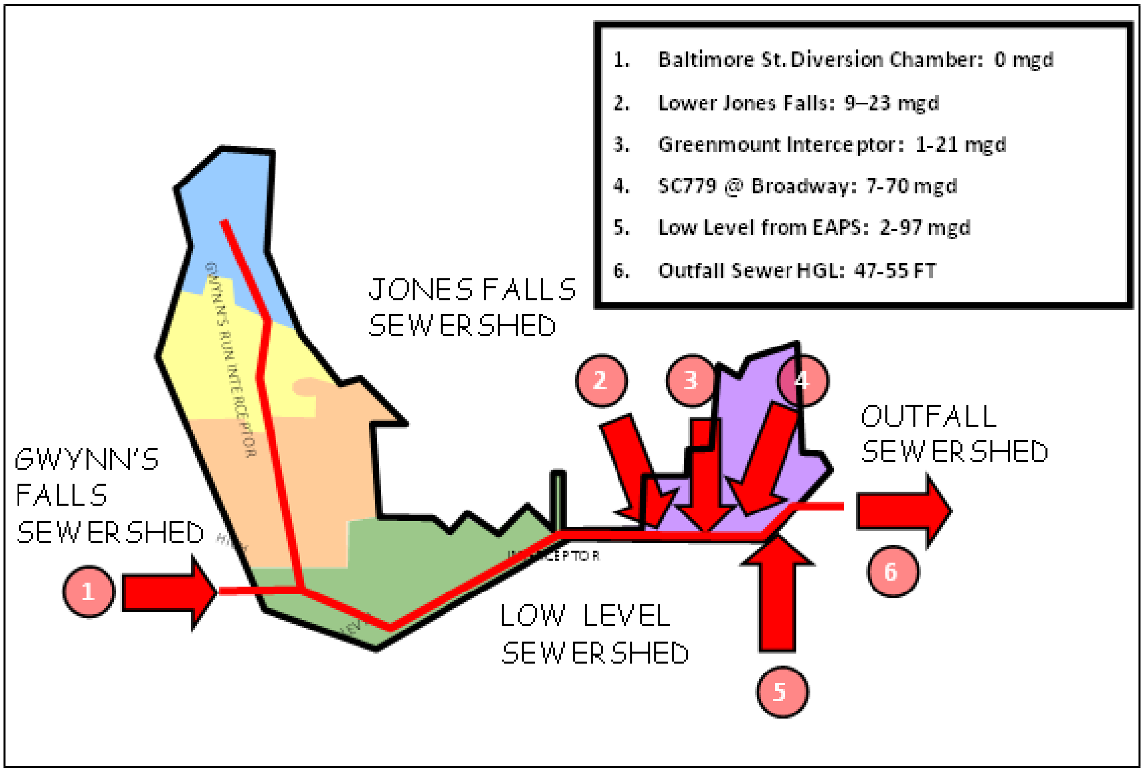


Figure 2.1: Inflows from other sewersheds into High Level Sewershed

Table 2.1: Dry and Wet Weather Flow Rate Range from HLSS Boundaries

Flow Sources	Flow Direction	Flow Meter Identifier	Pipe Size (in)	Monitored Dry weather flow range (MGD)	Monitored Peak wet weather flow (MGD)
Baltimore Street Diversion*	Into HLI	BHL1	33"	10 - 12.5	18
Jones Falls Interceptor	Into HLI	JFOUT	78"	5 - 15	33
Jones Falls Pump Station**	Into HLI	JFPS	60"	10 - 20	66
Outfall Sewershed	Into HLI	OUT05***	15"	N/A	N/A
Eastern Avenue Pump Station (EAPS)	Into HLI	OUT06	99"	10 - 40	76
HLSS and all the boundary flows	Out of HLI	TSHL01	144 (W) 129 (H)	70 - 90	170

* The diversion gate was closed on October 6, 2008 and no flow has been diverted to HLSS since then

** Flow meter is located near the Johns Falls pump station, upstream of the Jones Falls Force Main

*** OUT05 did not record depths/velocities properly due to high flow fluctuation from the EAPS

2.2 PARAGRAPH 8 PROJECTS

Three construction and rehabilitation projects, required under Paragraph 8 of the CD "Paragraph 8 Projects" were recently completed within the HLSS and are included in the baseline. The purpose of these projects is to eliminate SSOs from the engineered overflow structures. A listing of the Paragraph 8 projects is presented in Table 2.2.

SC812 significantly modifies the physical configuration of HLSS and hence the physical model. In order to accurately reflect the SC812 relief sewer in the hydraulic model, the HLSS team prepared two calibrated models. One model omits SC812 and the other model includes SC812, since it was put in service during the middle of the primary flow metering period (May 2006 – May 2007). Projects SC807 and SC831 were both incorporated into the model by reducing the pipe size by 0.5 inches and decreasing the pipe roughness to a range between 0.011 and 0.013.

Table 2.2. Paragraph 8 Projects in the High Level Sewershed

Sanitary Contract No.	Description	Completion Date	SSO Eliminated
SC812	Install 9,000' of 30" relief sewer parallel to Gwynn's Run Interceptor	Feb-07	106,107, and 130
SC807	Rehabilitate 22,000' of 8" to 30" pipe	Mar-03	55,56,57,60,63,126,127,128, and 131
SC831	Rehabilitate 12,000' of 8" pipe	Apr-08	N/A

2.3 REMAINING ENGINEERED SSOS

As shown in Table 2.2, most of the engineered SSO manhole locations have been eliminated in the HLSS since entry of the CD. Three known engineered SSOs remain active. These SSOs are listed in Table 2.3 below:

Table 2.3 Engineered SSO Remaining in the High Level Sewershed

SSO Number	Location	Receiving Waters	Manhole ID
132	Springdale Ave. and Hilton St.	Gwynn's Run Interceptor	S09UU_010MH
134	Liberty Heights Ave. and Ellamont Rd.	Gwynn's Run Interceptor	S11UU_016MH
135	Liberty Heights Ave. and Dennlyn Rd.	Gwynn's Run Interceptor	S11UU_008MH

The locations of all remaining active engineered SSOs are shown in Figure 1.1. In addition to the known engineered SSOs, the HLSS team has discovered two active overflow structures cross-connecting sanitary and storm.

Cold Spring Lane Overflow Structure

A 12-inch overflow pipe was found open at the intersection of Cold Spring Lane and Ayeardale Avenue. The overflow pipe is about 4 ft above the sanitary manhole invert, while the pipe intrudes into the storm sewer without a flap valve. Water can flow from sanitary to storm system or storm to sanitary system depending on the flow elevations in these two sewer systems.

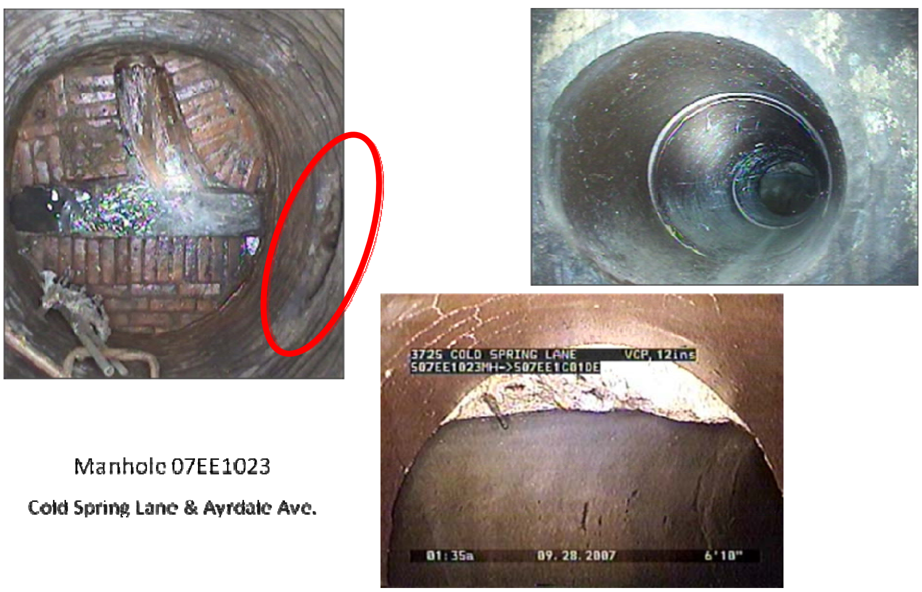


Figure 2.1.A: Pictures of the Cold Spring Lane Overflow Structures

Garrison Avenue Overflow Structure

A 15-inch overflow pipe was found at the intersection of Garrison Avenue and Queensberry Avenue. The overflow was verified during field inspection as an improper connection. The invert of the overflow pipe on the sanitary side is about 5-6 ft above the manhole invert and is lower than the bench of the storm drain manhole. Wastewater and storm water can flow either way between sanitary and storm sewers through the overflow, based on the hydraulic conditions in the two sewer systems.

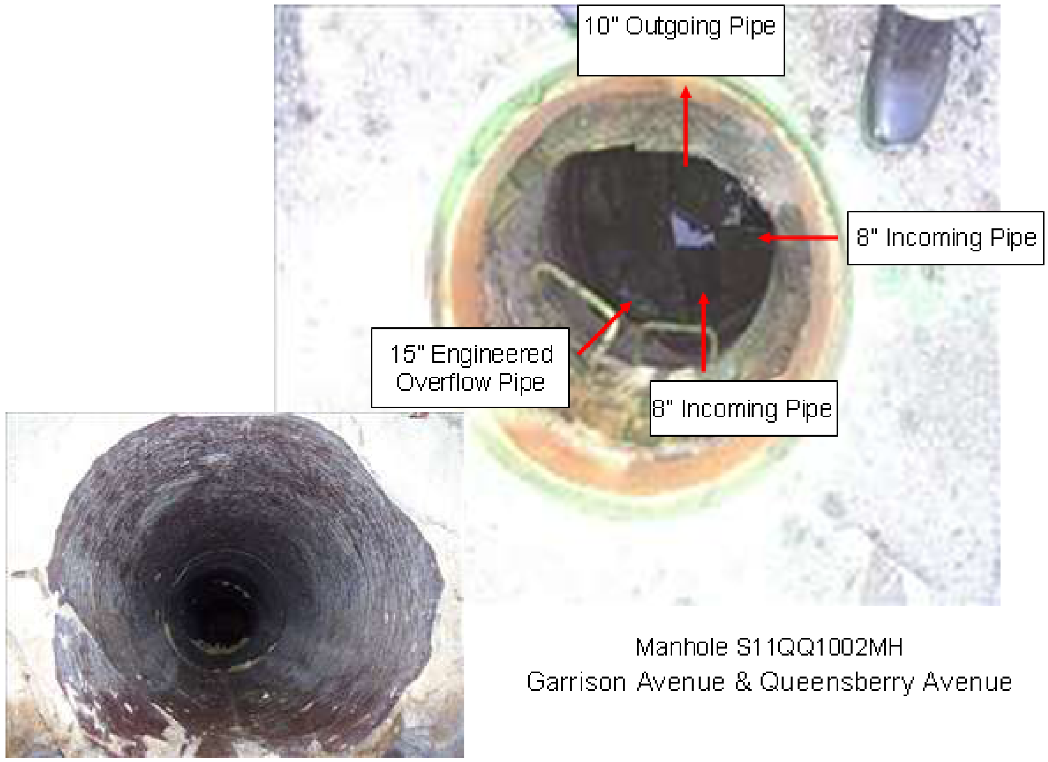


Figure 2.1.B: Pictures of Garrison Avenue Overflow Structure

At the two discovered overflow structure locations, the flows have been monitored at both the incoming pipe of the sanitary manhole and the overflow pipe to study how the overflow structures functioned during wet weather. The monitoring began on April 3, 2009 at the Cold Spring Lane overflow and April 23, 2009 at the Garrison Avenue overflow. No overflow event had been observed at either location according to the flow data compiled till May 10, 2009.

2.4 HYDRAULIC MODEL CALIBRATION

The hydraulic model of the HLSS was calibrated for both dry weather and wet weather flows using the monitoring data obtained during the primary monitoring period from May 2006 to May 2007. In the HLSS, 40 flow meters were used for model calibration and five meters were used to assign boundary conditions to the model. A detailed description of the model calibration is contained in the January 2009 *Model Development and Calibration Report*.

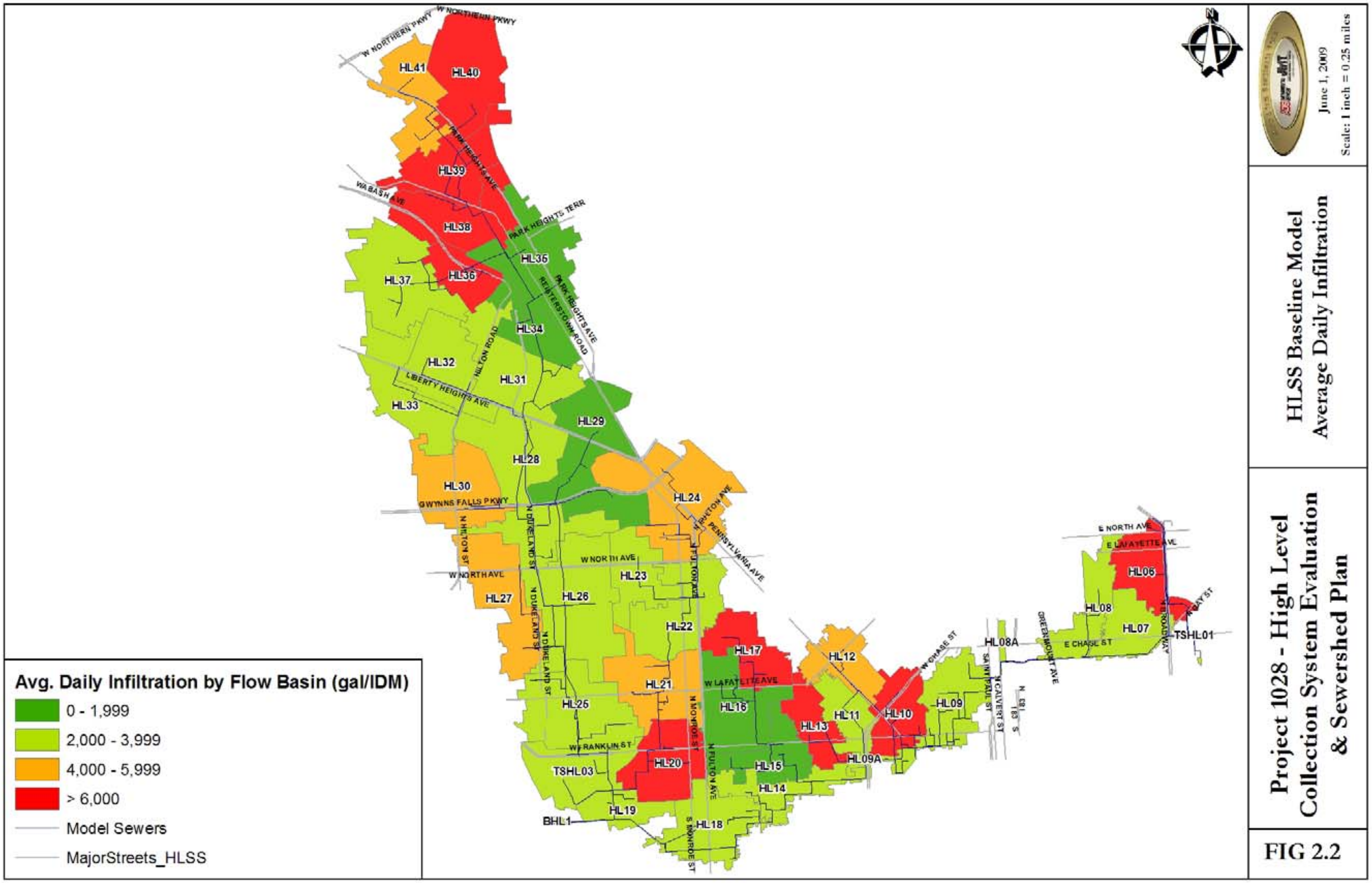


Figure 2.2: HLSS Baseline Model Average Daily Infiltration

2.4.1 Summary of Dry Weather Calibration

The two components of the dry-weather flow (DWF), average base sanitary flow (BSF) and groundwater base infiltration (BI), were first quantified during model calibration using Sliicer.com a City approved online inflow and infiltration analyzer developed by the ADS Environmental, Inc. Figure 2.2 shows the estimated average daily base infiltration for the HLSS. The BI evaluation was difficult for flow basins along the GRI and HLI for several reasons, including highly varying inflows from the upstream Ashburton Water Filtration Plant and/or larger gross flows compared to the net flow. All the flow basins along the HLI and four flow basins along the downstream portion of the GRI were aggregated separately to quantify the BI component.

Sliicer.com analyses yielded average daily DWF hydrographs for each monitoring basin for both weekdays and weekends. This data was then used to develop hourly diurnal peaking factors. This was done by first subtracting BI from the hourly values of the DWF hydrographs and then dividing by the average BSF.

Six events were selected among the three seasons of study (Summer 2006, Winter 2007 and Summer 2007) for both pre and post-SC812 conditions to support the DWF calibration. The primary rationale for selecting those events was to choose dry periods with no rainfall for at least 48 hours prior to the event so that there would be little or no residual moisture that might affect infiltration during these periods. The duration of events ranged from 5 to 12 days in order to characterize the possible variations between the weekday and weekend water usage patterns, in accordance with the BaSES guidelines.

For dry weather, the pipe roughness was primarily used to calibrate the model for depth and velocity at each flow meter location. The adequacy of model calibration was assessed using time-series plots of simulated and observed flow, depth and velocity compared at each flow meter and the average flow rate on a system-wide basis. The model performance, in terms of the correlation between monitored and modeled data, was very good at most of the locations for flow rate, depth and velocity.

2.4.2 Summary of Wet Weather Calibration

Prior to conducting the wet weather flow calibration, Sliicer.com was used to determine the RDII severity in HLSS. The severity was expressed in terms of a capture coefficient (R-value); which is defined as the fraction of rainfall volume that entered the sanitary system for a given total rainfall volume for each flow basin. The RDII quantification during wet weather in Sliicer.com is extremely challenging under one or more the following conditions:

(a) net basin flow is less than 20% of gross flow,

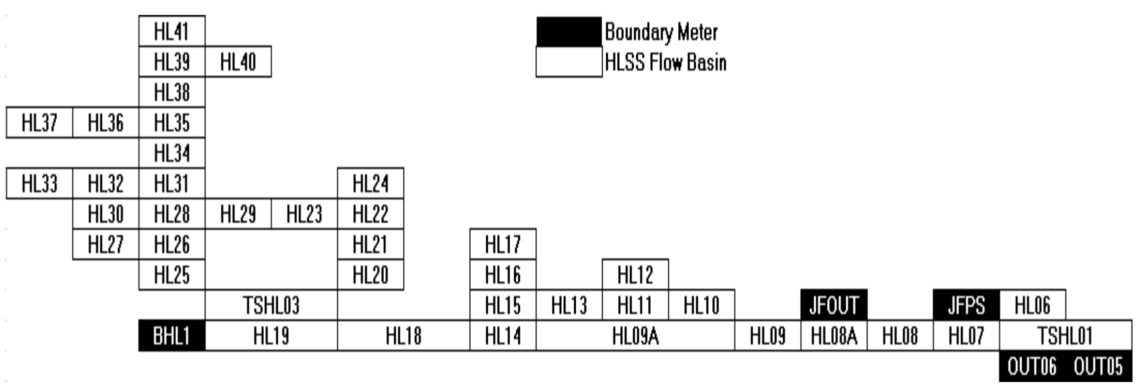
(b) boundary flow is significant, and

(c) irregular and undocumented pump station discharges.

In HLSS, the RDII severity could not be quantified for three flow basins (HL25, HL26, and TSHL03) along the GRI due to irregular and undocumented pump discharges from the Ashburton Water Filtration Plant and its Wash Water Lake. RDII also could not be accurately quantified for six flow basins (HL07, 08, 09, 14, 18, and 19) along the HLI due to their large net-to-gross flow ratios and the large boundary flows from the Jones Falls and Low Level sewersheds. For these interceptor flow basins, the R-values were initially determined based on the average value for a number of nearby HLSS basins and further adjusted during model calibration.

Except for the interceptor flow basins, the capture coefficient was calculated for both summer and winter seasons in Sliicer.com. Figure 2.3 shows the capture coefficient values for each flow basin. This Figure was color coded from light blue to dark blue based on the severity of I/I, as reflected by the increasing values of the capture coefficient. Two observations may be made based on the HLSS data analysis:

- RDII was more severe for flow basins contributing to the upstream portion of GRI
- RDII was more severe in winter than in summer for the entire HLSS



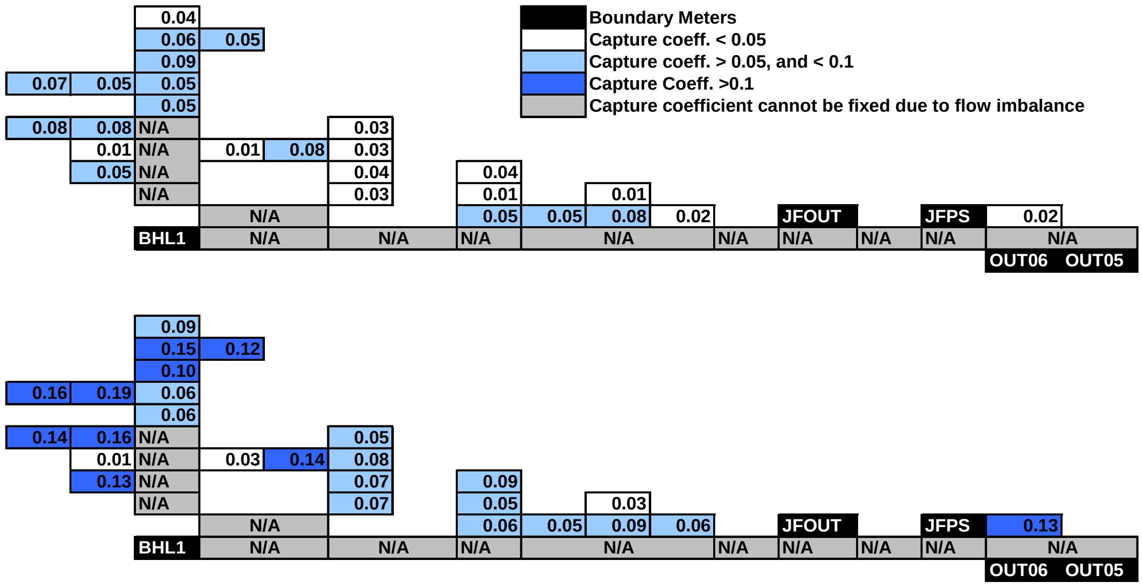


Figure 2.3: Processed Summer (above) and Winter (below) Capture Coefficients in HLSS

In order to simulate wet weather flow the SWMM RUNOFF routine available within InfoWorks CS was used as a synthetic storm hydrograph generator. Simulating the rainfall-dependent infiltration and inflow (RDII) using SWMM RUNOFF requires the specification of catchment characteristics that result in correct estimation of the RDII. The parameters needed are: area, R-value, depression storage, width, slope, and an overland flow routing coefficient.

Wet weather flow calibration was conducted separately for the summer 2006, winter 2007, and Post-SC812 periods to reflect the significant seasonal RDII severity changes and the change due to the installation of a 30" relief pipe along the GRI (SC812). Calibration was conducted based on the 29 "global" storms for which the radar rainfall data were provided by the City. The runoff routing value was used as the primary calibration parameter to achieve the desired RDII volumes, while the catchment width and slope were used as supplemental parameters to achieve the desired time-to-peak and peak flow characteristics. Capture coefficient and depression storage, derived from Sliicer.com, were used as fixed parameters in the RDII analysis. Calibration results were reviewed using time-series plots for flow, depth, and velocity and evaluated using model calibration criteria suggested in BaSES manual.

In order to assess whether the calibrated model satisfied the criteria for each metered location, the HLSS team also evaluated goodness-of-fit plots to compare the simulated and observed values for peak flow, flow volume, peak depth, and peak time. Figure 2.4 shows an example of goodness-of-fit plots for flow meter HL07 located closer to the downstream end of the HLI. The calibration criteria for peak flow rate, volume, and surcharge depth are represented as grey dashed lines (on either side of the green 45-degree line that represents a perfect correlation between the two). This provides a visual check to assure that the model results meet the criteria for most of the storms.

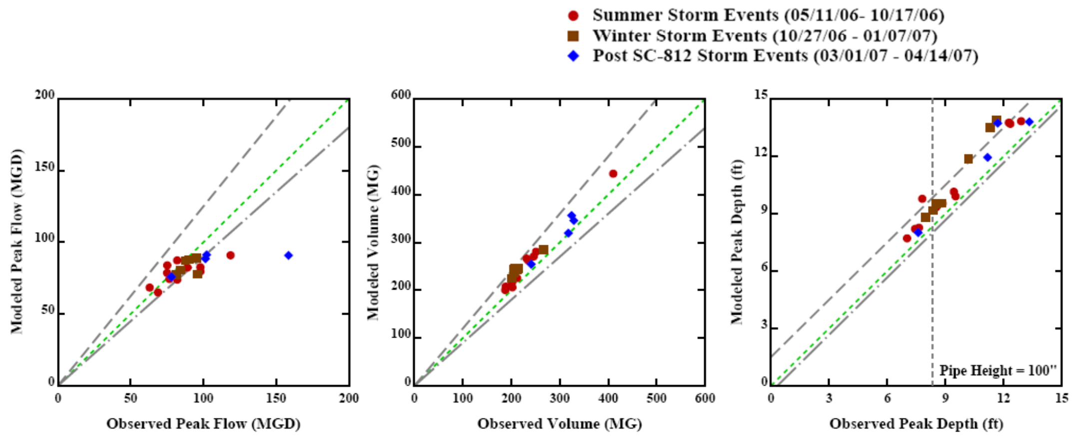


Figure 2.4: Goodness-of-fit Plots at Flow meter HL07

The adequacy of wet weather flow calibration was assessed by the HLSS team using two additional metrics: historical SSO locations and maximum HGL. Two large storms, one on July 5th, 2006, and the other on November 16th, 2006, were selected from the monitoring period. The twenty-four hour intensity for both storms approximated a 2-year return frequency. The potential SSO locations revealed from the simulation results of these two storms were compared with the historical SSO locations. The simulated maximum HGLs along HLI were also compared with the observed data at each flow metering location. The model results correlated very well with observed data in the entire system.

Finally, the calibrated summer and winter models were combined into a unified model using the median-R capture coefficient as required by the City. Figure 2.5 shows the median R capture coefficient in the HLSS. The combined median-R model was further fine-tuned using several major global storms so that the model could also accurately represent the system behavior during intense storm conditions used for the baseline and alternative analysis.

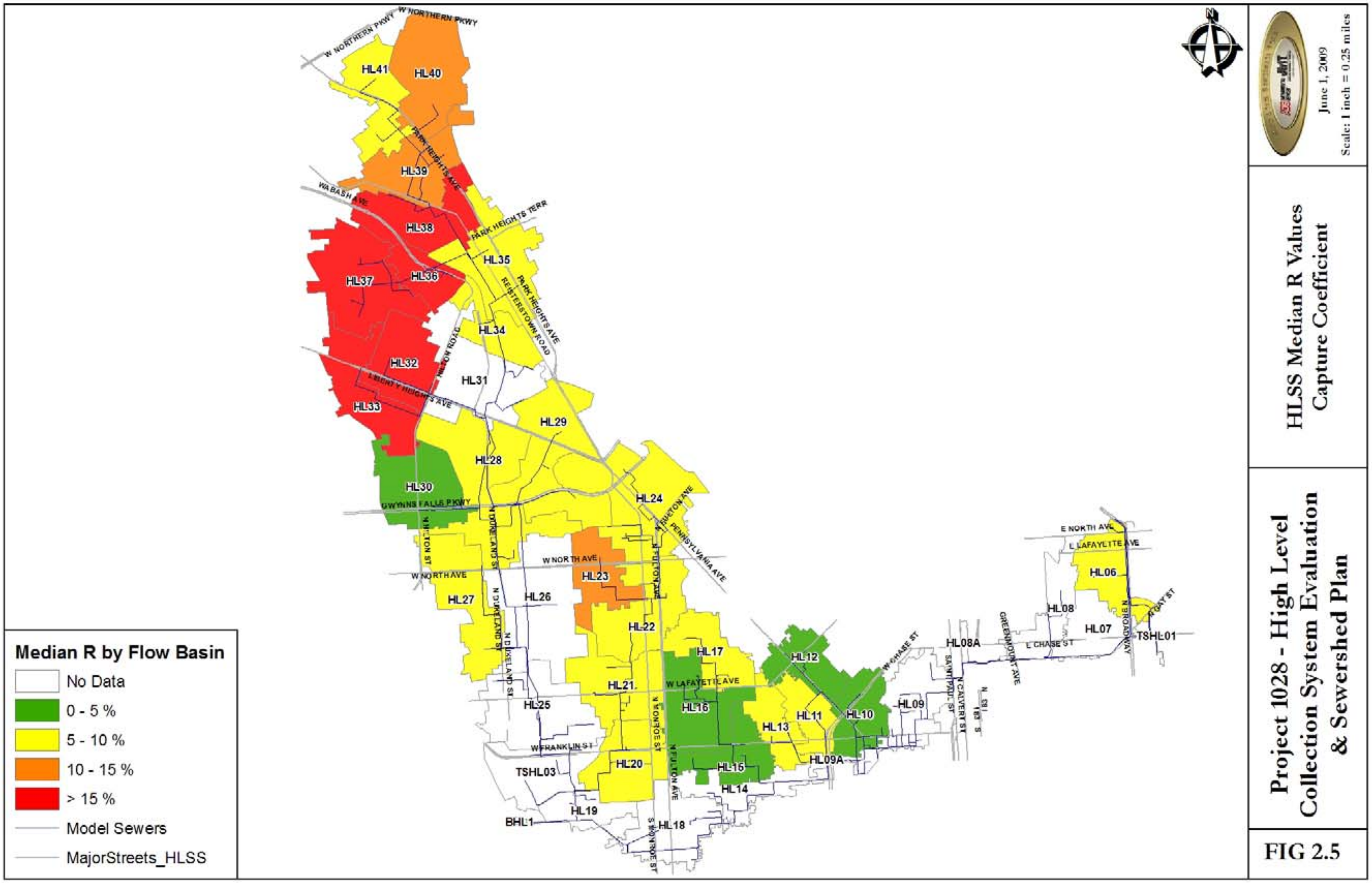


Figure 2.5: HLSS Median-R Value Capture Coefficients

SECTION 3

BASELINE ANALYSIS AND CAPACITY ASSESSMENT

3.1 GENERAL

Baseline conditions are defined by the City to represent the system conditions after the completion of Paragraph 8 projects in the sewershed. The calibrated hydraulic model was updated to simulate the baseline conditions, based on the Paragraph 8 information available in various as-built drawings. The model was run for both dry weather and wet weather flow conditions to identify areas of the HLSS collection system which lacked adequate capacity to convey the projected flows under the baseline conditions. The wet weather storm events that were modeled include a three-month storm having a duration equal to the time of concentration for the sewershed (2.5 hours); and 1-, 2-, 5-, 10-, 15-, and 20-year, 24 - hour storms.

3.2 BOUNDARY CONDITIONS FOR BASELINE ANALYSIS

The HLSS has five major boundary flow sources: Baltimore Street Diversion (BSD), Lower Jones Falls Interceptor (LJF), Greenmount Interceptor, Jones Falls Force Main (SC779) and the Eastern Avenue Force Main (EAFM). In addition to these, the Ashburton Water Filtration Plant (WFP) discharges up to 5 MGD of flow into the Gwynn's Run Interceptor. Some of these boundary flows are larger than the flow generated from the HLSS in both dry and wet weather conditions. This highlights the importance of boundary conditions for the Baseline Analysis. Initially, the Technical Management Team SC 1015 had provided boundary conditions generated by the macro-sewershed model; however some of these boundary conditions did not reflect the realistic hydraulic conditions along the HLI. Therefore, the SC 1015 Team re-produced boundary conditions for the HLSS team after they combined the various sewershed models, also known as the "micro models", into a single City-wide model, called the "macro model". Individual model outputs were rerun with the macro model for the Baseline conditions. In this section, the new HLSS boundary conditions are discussed in detail.

3.2.1 Ashburton WFP Inflows

Ashburton WFP is the largest wastewater discharger for the HLSS sanitary sewer system. This plant discharges its wastewater from the sedimentation basin and filter backwash water into the GRI. This is the most significant single point discharge into the GRI. The peak discharge rate to GRI exceeded 10 MGD due to the use of temporary pump stations located at the Washwater Lake, which was under rehabilitation (WC1143) and subsequently, overwhelmed the capacity of GRI. After the rehabilitation project was completed in April 2009, the plant's discharge (controlled by a modular valve) does not exceed 5 MGD per set the design criteria. Although the plant discharge is intermittent, a conservative estimate of a continuous 5 MGD inflow during the 24-hour storm duration was applied for the Baseline simulations.

3.2.2 Boundary Conditions from Adjacent Sewersheds

The HLSS interacts with other sewersheds at several locations on the HLI. The HLI receives significant discharge into GRI from the Ashburton WFP and inflows from Jones Falls and Low Level Sewersheds at the lower end which then discharges into the Outfall sewershed through the HLI (Figure 1.1). Flow or depth boundary conditions were provided to the HLSS team for each of these locations by the City's Technical Management Team, SC 1015. Table 3.1 summarizes the boundary conditions used for the baseline simulation, which are discussed further in this section.

Table 3.1: Boundary Conditions for High Level Sewershed

Location	Type	Range
Baltimore Street Diversion	Flow	0* (MGD)
Lower Jones Falls Interceptor	Flow	9 - 23 (MGD)
Greenmount Interceptor	Flow	1 - 21 (MGD)
SC779 (JF and Stony Run pump stations)	Flow	9 - 70 (MGD)
Eastern Avenue Pump Station (EAPS)	Flow	2 - 97 (MGD)
Downstream end of HLSS (TSHL01)	Depth	47 - 55 (ft-Elevation)

* it is assumed that no flow is diverted from the Baltimore Street Diversion to the HLI

Baltimore Street Diversion (BSD)

The City provided the following guidance on the baseline condition for the BSD.

> The BSD structure is designed to be normally closed, meaning that ALL flow is diverted to the Patapsco WWTP and NO flow into the Back River WWTP via the HL Interceptor. Diversion of flows to the HL interceptor would happen only under emergencies or extraordinary circumstances. However, over the past couple of years, the City has been diverting approximately 20 MGD to the HL interceptor due to the repairs to the Southwest Diversion Pressure Sewer. Those repairs have been completed, and the BSD structure has been in normal operation since October 7, 2008, thus sending all flows to the Patapsco WWTP.
>
> Therefore, the HL Team must assume no flow being transferred to the HL Interceptor for baseline analysis. However, as the HL Team continue to analyze the

different storms, it may be beneficial to evaluate whether there is any capacity in the HL Interceptor that would allow some flow to be transferred.

Per the guidance provided by the City, the HLSS team assumed zero flow will be transferred to the HLI for all the baseline simulations. Based on the HGL conditions along the HLI from model simulation and huge inflows from the Jones Falls and Low Level sewersheds, the HLSS team does not believe that there is any capacity in the HLI that would allow flow transfer from the BSD.

Lower Jones Falls Interceptor (LJF)

Figure 3.1.A shows the provided time-series inflow boundary conditions of the LJF Interceptor for dry weather and various design storm conditions. For the 1-year and larger storm events, the flow increases approximately to 23 MGD within the first 12 - 15 hours and then decreases dramatically to approximately 7 MGD due to surcharging at the High Level Interceptor.

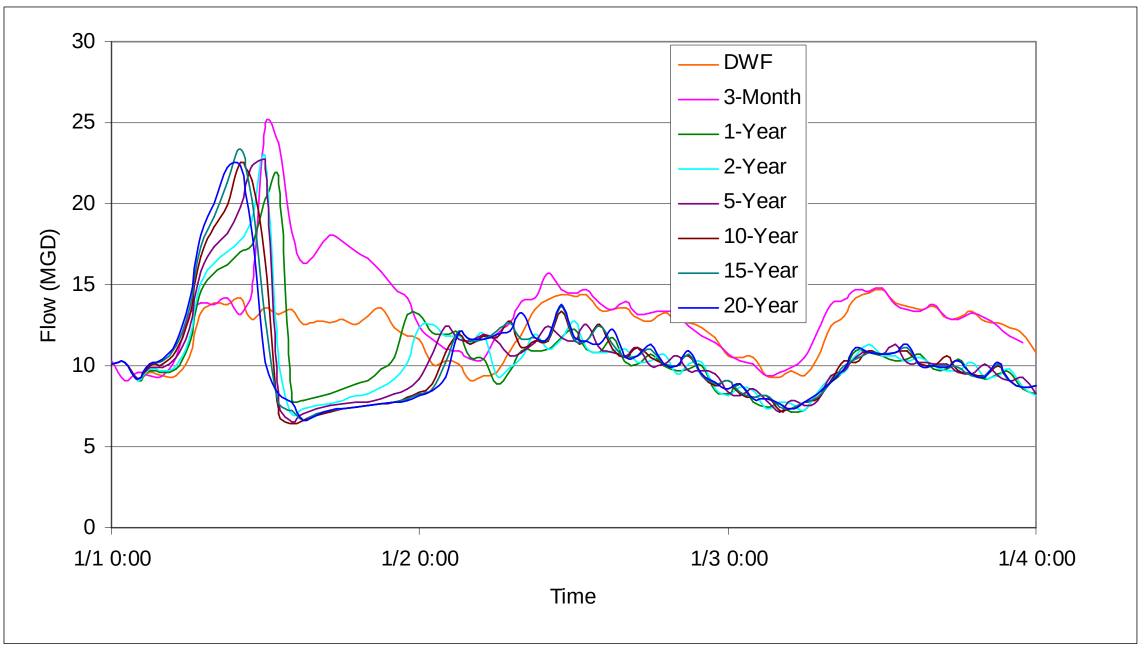


Figure 3.1.A: Inflow boundary conditions for the Lower Jones Falls Interceptor

Table 3.1.A. Minimum and maximum boundary flows from Lower Jones Falls Outfall

	DWF	3-month	1-year	2-year	5-year	10-year	15-year	20-year
Min. (MGD)	9.1	9.1	7.2	7.1	6.5	6.5	6.6	6.6
Max. (MGD)	14.7	25.0	21.7	22.7	22.6	22.6	23.4	22.3

Greenmount Interceptor

Greenmount Interceptor coming in from the Jones Falls Sewershed is a newly constructed relief pipe connecting to the HLI approximately 1,200 feet east of the LJF interceptor. The Greenmount Interceptor has been in service since May 2008. Figure 3.1.B shows the provided time-series inflow boundary conditions for the Greenmount Interceptor during dry weather and each design storm conditions, and their maximum and minimum rates are summarized in Table 3.1.B

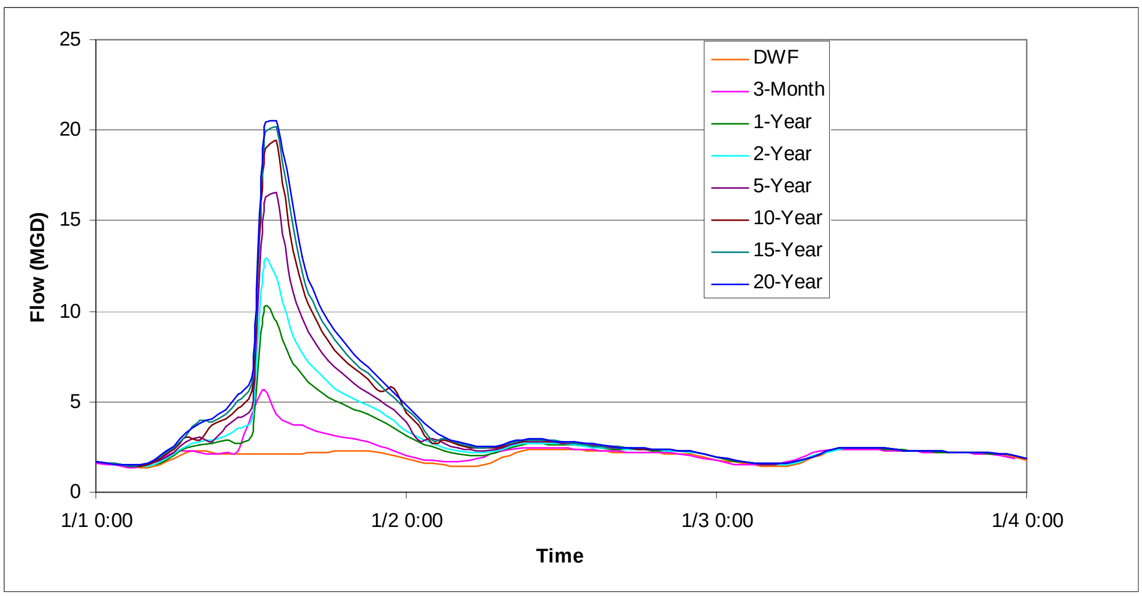


Figure 3.1.B: Inflow boundary conditions from Greenmount Interceptor

Table 3.1.B Minimum and maximum boundary flows from Greenmount Interceptor

	DWF	3-month	1-year	2-year	5-year	10-year	15-year	20-year
Min. (MGD)	1.4	1.4	1.4	1.4	1.4	1.5	1.5	1.5
Max. (MGD)	2.4	5.6	10.2	12.8	16.6	19.5	20.2	20.5

SC779

SC779 is relief sewer that carries flow from the Jones Falls Pump Station Force Main and Stony Run Pump Station Force Main. The DWF has a diurnal pattern because the Stony Run Pump Station only activates during elevated flow conditions. During the normal DWF conditions, this pump station is not used and the flow bypasses this station. Refer to Figure 3.1.C for the inflow hydrographs at this boundary condition, and Table 3.1.C for maximum and minimum flow rate summary.

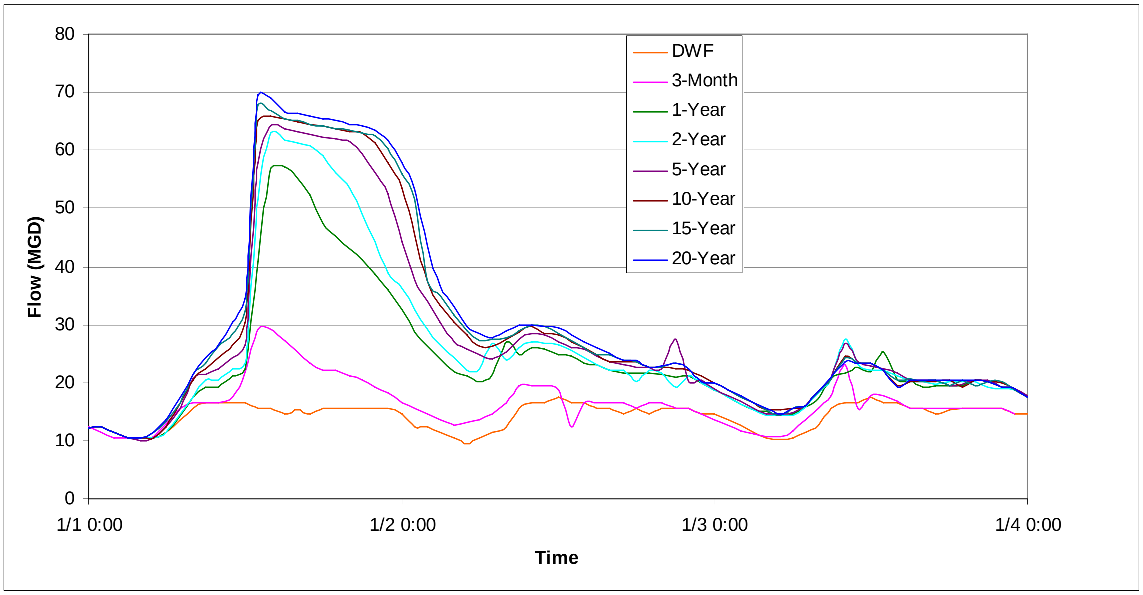


Figure 3.1.C: Inflow Boundary Conditions from SC779

Table 3.1.C: Minimum and Maximum Boundary Flows from SC779

	DWF	3-month	1-year	2-year	5-year	10-year	15-year	20-year
Min. (MGD)	9.5	10.5	10.5	10.5	10.1	10.5	10.5	10.5
Max. (MGD)	17.5	29.3	57.1	62.9	64.2	65.9	67.8	69.5

Eastern Avenue Pump Station Force Main (EAPS FM)

Figure 3.1.D shows the provided inflow pattern from the EAFM. Peak wet weather flow from the EAFM is approximately 95 MGD, while the peak DWF is approximately 40 MGD. The fluctuating pattern after each storm event shows that the pump discharge pattern every 3-6 hours.

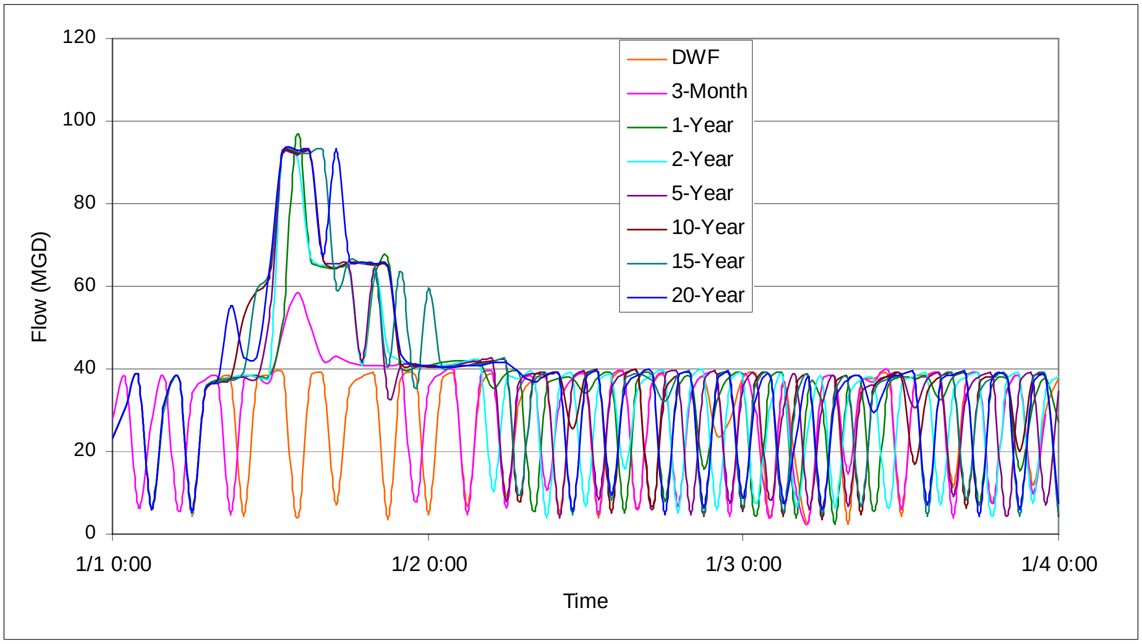


Figure 3.1.D Inflow boundary conditions from the Eastern Avenue Force Main

Table 3.1.D: Minimum and maximum boundary flows from Eastern Avenue Force Main*

	DWF	3-month	1-year	2-year	5-year	10-year	15-year	20-year
Min. (MGD)	2.5	3.0	2.4	3.8	4.0	3.4	4.2	4.4
Max. (MGD)	39.1	58.5	96.9	92.3	92.5	92.5	93.0	93.4

* Boundary condition was not provided for dry weather flow analysis

3.3 DRY WEATHER CAPACITY ASSESSMENT

An assessment of dry weather capacity for the baseline condition was conducted. The DWF simulation was run for a one week duration using weekday and weekend diurnal patterns for each flow basin. The revised inflows provided by 1015 and developed using the macro model was used for the dry weather baseline simulation.

Figure 3.2 groups the modeled pipes by the percentage of filled pipe capacity during peak hourly DWF. The model shows no SSOs in the HLSS at this peak DWF condition. However, there are several pipe segments that are surcharged (shown in red in the map). Some of the surcharged pipes are small pipe segments that connect to the larger interceptor at approximately the same invert as the interceptor. Therefore, when the level in the interceptor exceeds the diameter of the small pipes, those pipe segments are shown as surcharged. Two pipe segments are surcharged along the GRI due to the 5 MGD discharge from the Washwater Lake and heavily accumulated sedimentation along the GRI. If heavy cleaning is conducted along the GRI, the surcharges will be reduced or completely resolved.

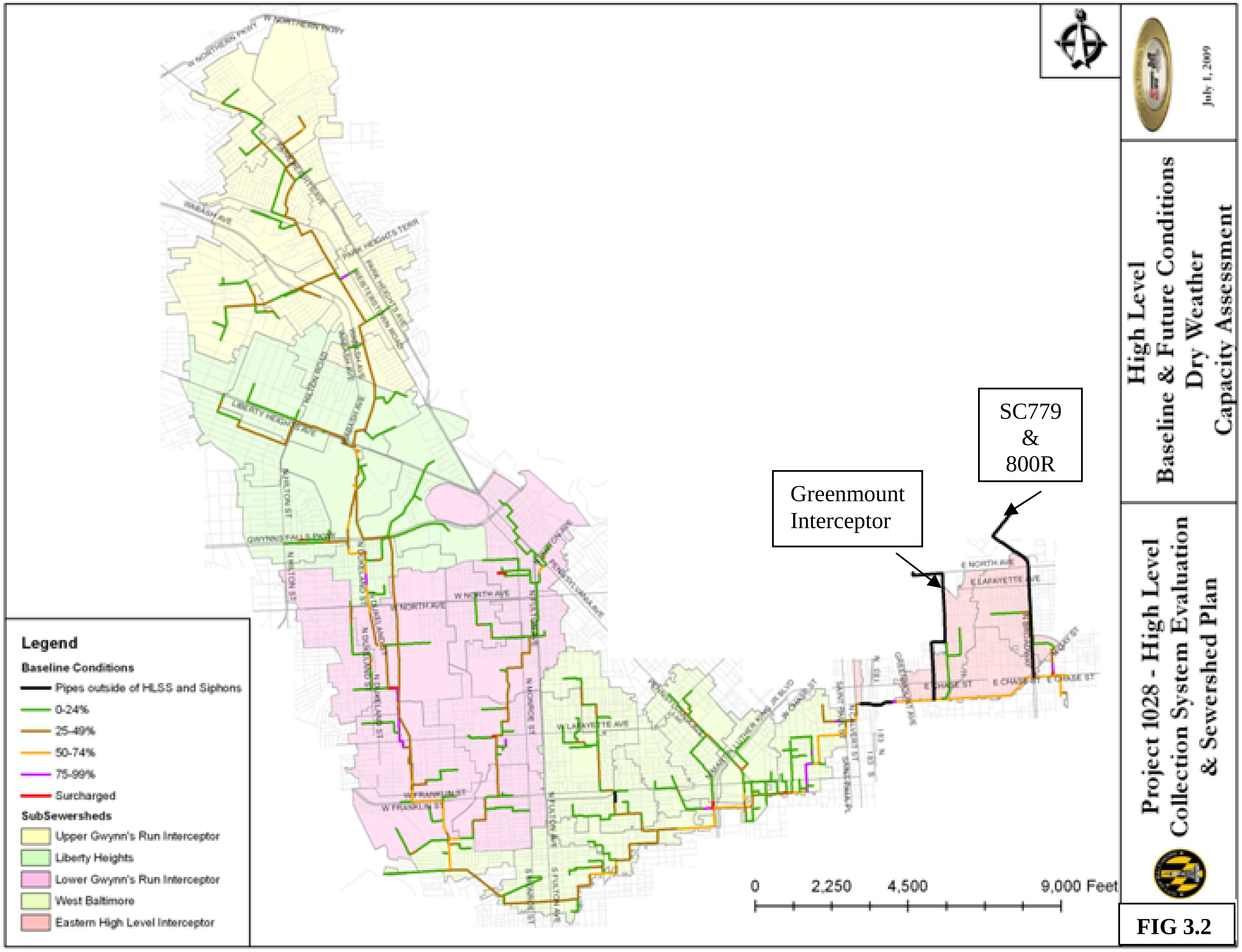


Figure 3.2: High Level Baseline & Future Conditions Dry Weather Capacity Assessment

The pipes that are above 75% full during peak DWF are highlighted in purple and above 50% full in yellow in Figure 3-2. Most of the surcharges occurring between the HLI inverted siphon until the end of the HLI are due to sediment accumulation in the downstream portion of the HLI. The sediment depths range from one quarter to half of the pipe diameter, or pipe height in the non-circular pipe cross-sections, along the HLI.

3.4 WET WEATHER CAPACITY ASSESSMENT

3.4.1 Storm Events

Seven design storms were analyzed to assess the sewer capacity limitations during wet weather periods. These design storms include a three-month storm with a duration equal to the time of concentration for the sewershed (2.5 hours for the HLSS) and 1-, 2-, 5-, 10-, 15-, and 20-year, 24-hour duration storms. The storm distribution chosen for analysis is the NOAA Atlas 14/NRCS distribution. The storm depths for the seven design storms are as follows:

Table 3.2: Design Storms - Rain Depth (inches) and Peak Intensity (inch/hour)

Design Storms	Rain Depth (inches)	Peak Intensity (in/hour)
3 – Month, 2.5-Hour	1.11	1.3
1 – Year, 24-Hour	2.67	2.2
2 – Year, 24-Hour	3.23	2.6
5 – Year, 24-Hour	4.15	3.2
10 – Year, 24- Hour	4.97	3.6
15 –year, 24-Hour	5.41	3.7
20 Year, 24-Hour	5.82	4.0

Figure 3.3 shows the hyetograph of the provided design storms along with the actual observed peak intensity of three major storms that occurred during the model calibration period as references. The peak intensities of 5-year and the larger storms are much greater than the peaks of existing storms used during model calibration. Thus, it can be concluded that the model was calibrated with up to 2-year design storm severity, and the baseline simulation results with 5-year and the larger events could have some uncertainties. However, those should be very useful to compare and select cost-effective SSO mitigation

plans, considering the extensive model calibration involving over 40 meters and 29 global storms.

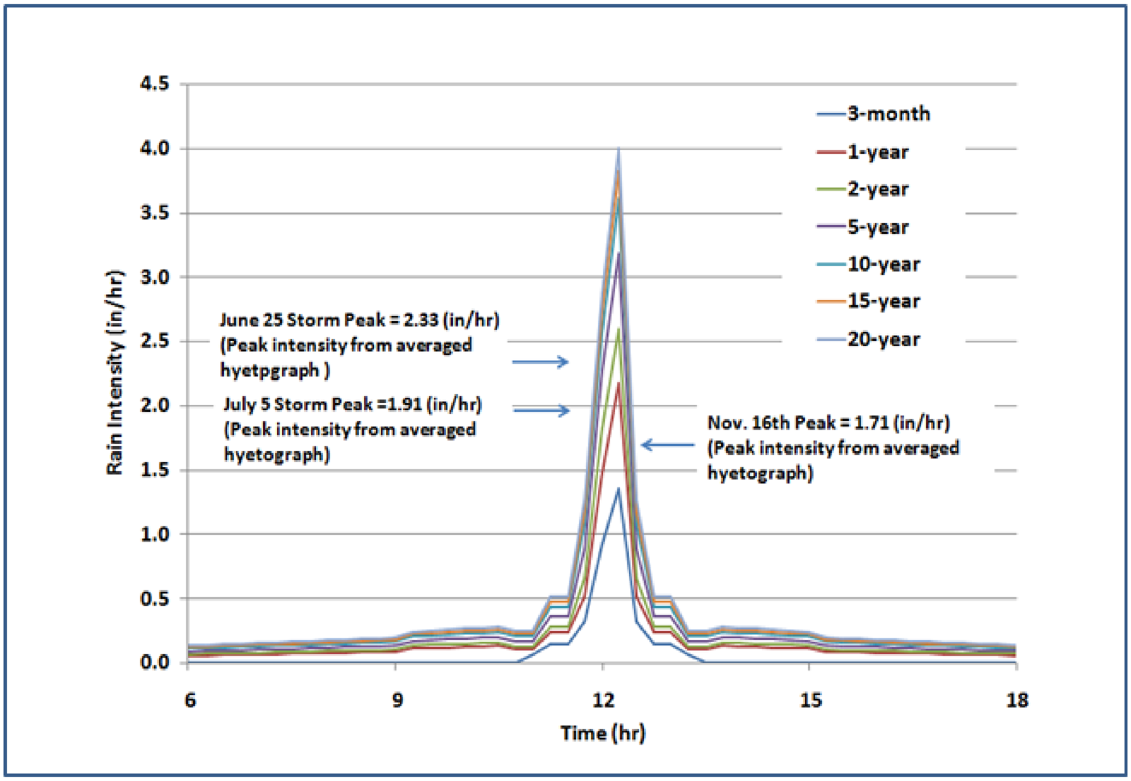


Figure 3.3: Hyetograph of 3-month, 1-, 2-, 5-, 10-, 15-, and 20-year Design Storms used for the Baseline Analysis

3.4.2 Return Period Analysis

One of the CD requirements is to run a Return Period Analysis (RPA) on the seven design storms. The InfoWorks' built-in RPA utility was used to compare the surcharge state in pipes and flooding results of the design storm runs in order to determine the minimum size storm required to surcharge or flood (cause an overflow) a pipe segment, along with the estimated flood volume. The results of the baseline flooding RPA are presented for each sub-sewershed in Figure 3.3.A (Upper GRI), Figure 3.3.B (Liberty Heights), Figure 3.3.C (Lower GRI), and Figure 3.3.D (West Baltimore and East HLI).

3.4.3 Predicted SSO Volumes

Under the DWF and 3-month storm conditions, there are no SSOs in the HLSS; but the model has predicted SSOs for all the other storm conditions analyzed. Table 3.3 shows the total SSO volumes through the manholes and each remaining active engineered overflows for the 1, 2, 5, 10, 15, and 20-year design storms.

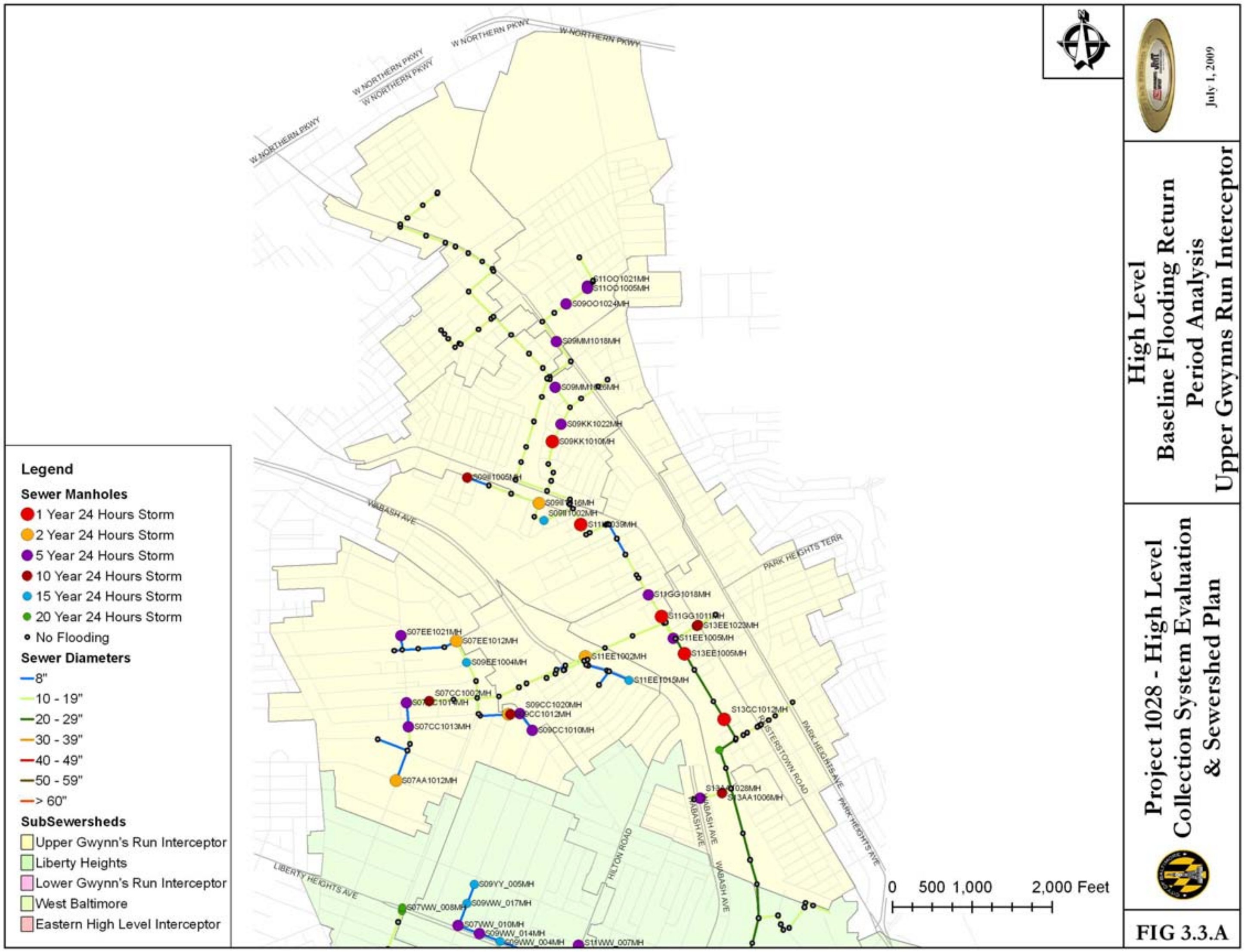


Figure 3.3.A: High Level Baseline Flooding Return Period Analysis Upper Gwynn's Run Interceptor

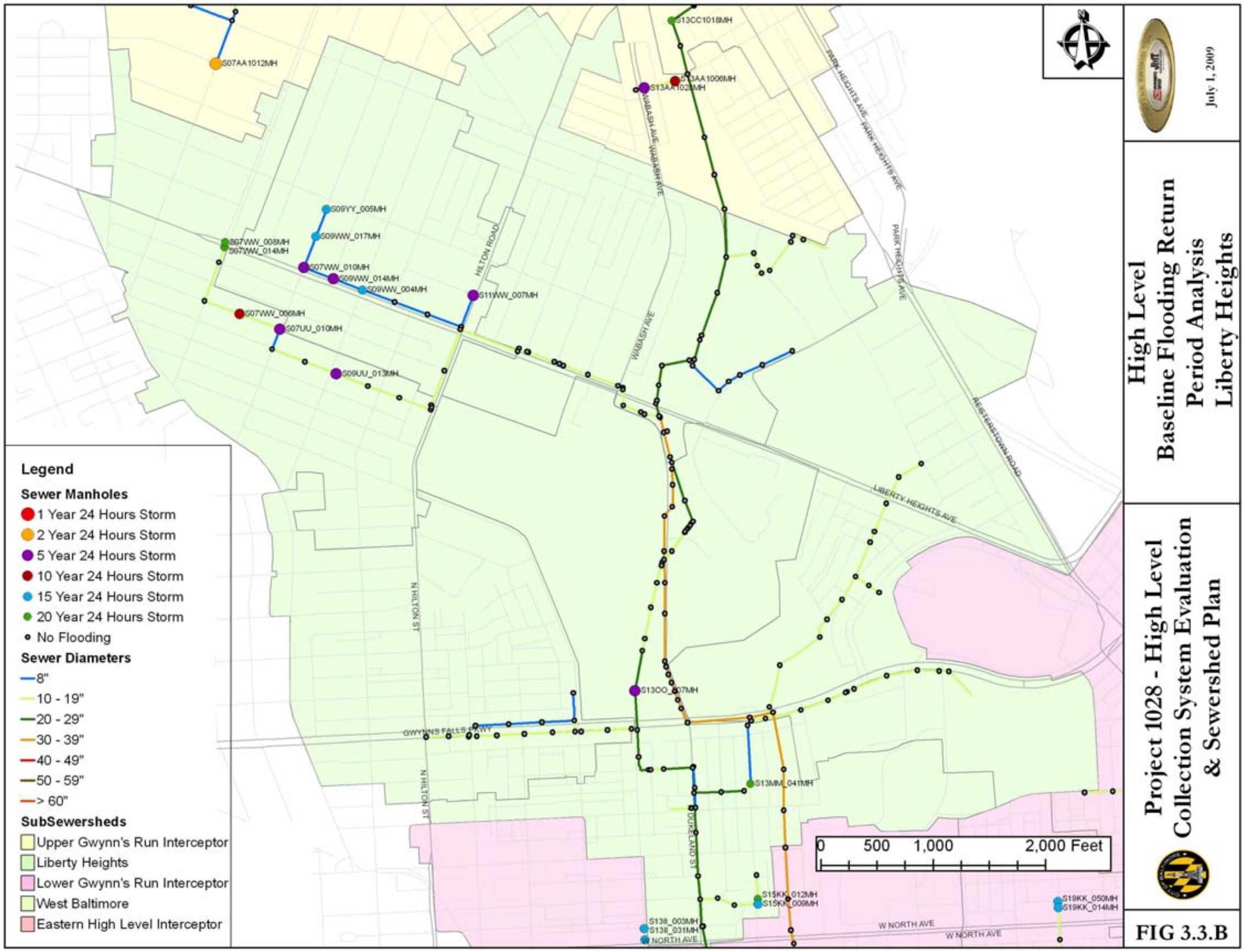


Figure 3.3.B: High Level Baseline Flooding Return Period Analysis Liberty Heights

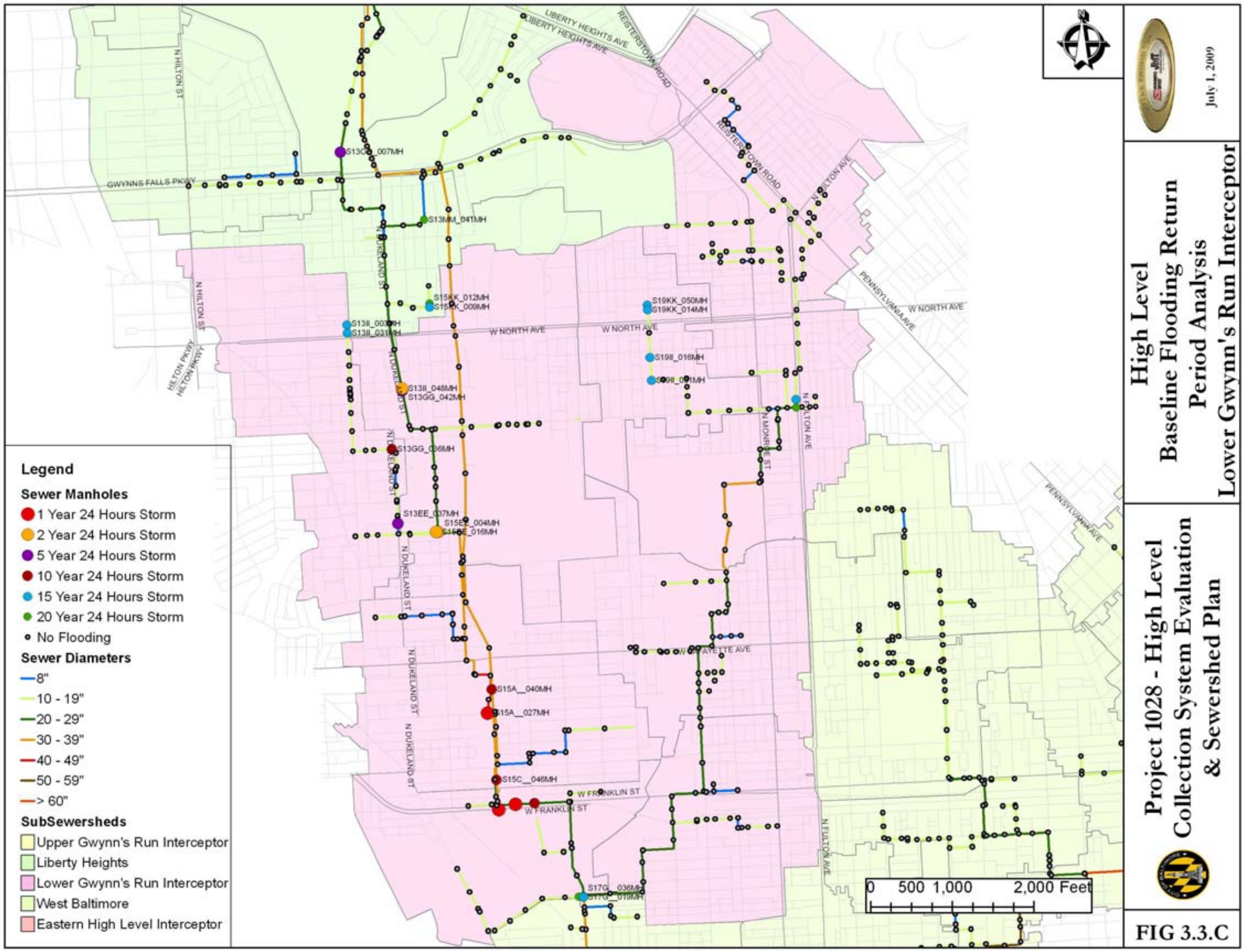


Figure 3.3.C: High Level Baseline Flooding Return Period Analysis Lower Gwynn's Run Interceptor

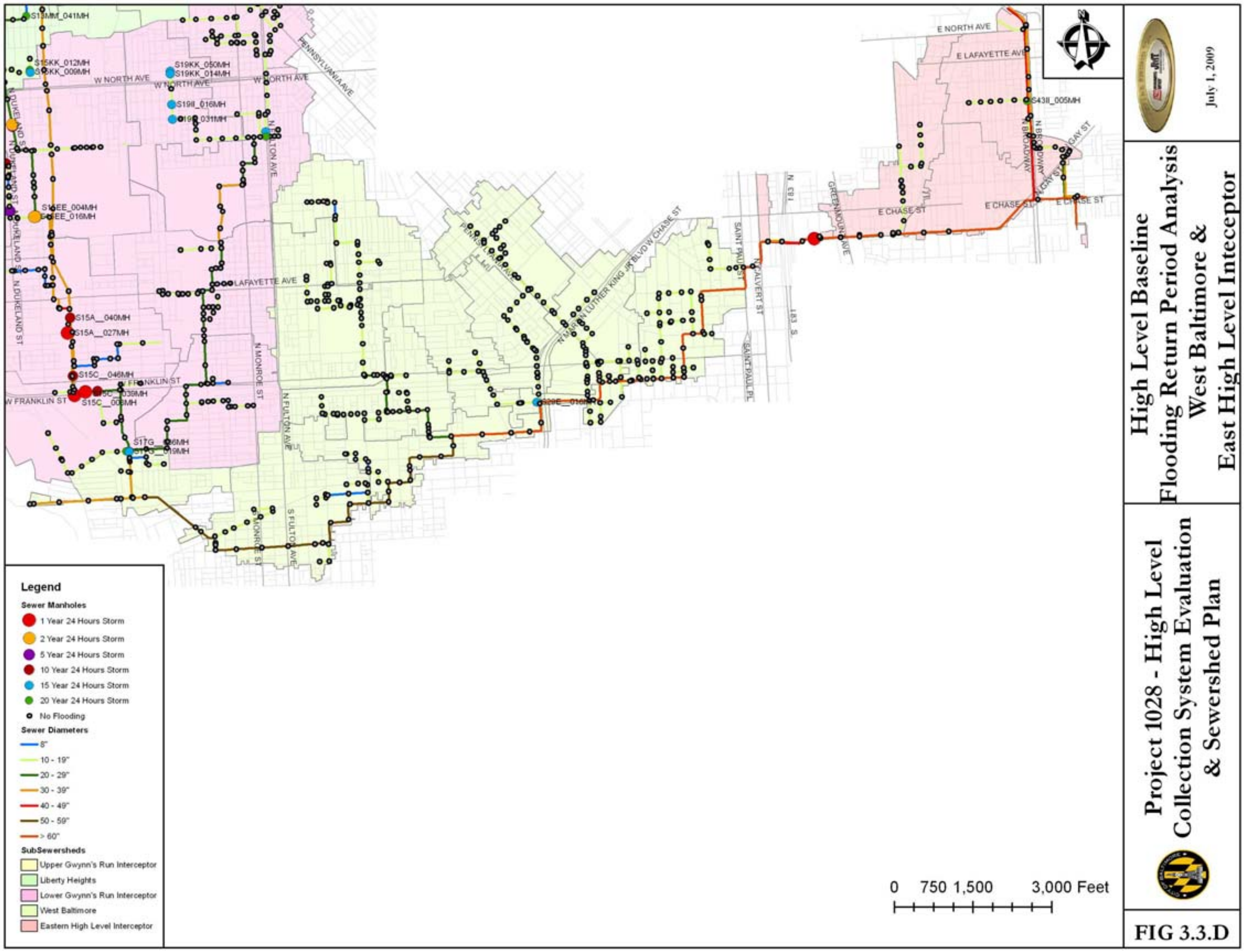


Figure 3.3.D: High Level Baseline Flooding Return Period Analysis West Baltimore & East High Level Interceptor

Table 3.3: Baseline SSO Volumes

Event	Manholes (MG)	SSO132 (MH)	SSO134 (MH)	SSO135 (MH)	Garrison (MH)	Cold Spring (MH)
3-Month, 2.5-Hour	0.00	0.00	0.00	0.00	0.00	0.00
1-Year, 24-Hour	2.89	0.13	0.12	0.00	0.00	0.01
2-Year, 24-Hour	5.72	0.34	0.18	0.00	0.00	0.04
5-Year, 24-Hour	10.05	0.70	0.25	0.00	0.04	0.09
10-Year, 24-Hour	14.15	0.98	0.31	0.00	0.08	0.12
15-Year, 24-Hour	16.43	1.13	0.33	0.00	0.11	0.14
20-Year, 24-Hour	18.43	1.26	0.36	0.00	0.14	0.16

Flooded manhole locations and the corresponding flood volumes are discussed below in detail for each sub-sewershed specified in Section 1.

Upper Gwynn's Run Interceptor

Table 3.4.A lists the overflow manholes for each design storm and Figure 3.3.A depicts the location of overflow manholes for the Upper GRI area. Along the GRI, there are several flooded manholes caused by a 1-year storm. This is due to the high RDII severity in the area served (Figure 2.2) and limited capacity of this interceptor relative to the large flows.

There are several overflow locations in HL36 and 37 for the 2-year storm. Because of the heavy loading of RDII into the drain node of each subcatchment in the simulation, in reality, some of these flooded manholes may not be experiencing overflows. The model was calibrated at each flow meter location, and the surcharge and SSO conditions at upstream manholes did not have any relevance during the calibration process which likely caused some manholes to erroneously overflow. The HLSS team paid careful attention to these overflow manholes during the alternatives development process in order to avoid recommending over-conservative solutions.

Liberty Heights

Table 3.4.B lists all overflow manholes for each design storm and Figure 3.3.B depicts the location of overflow manholes for the Liberty Heights area. There are only a few overflow manholes in HL32 and 33 areas resulting from large storms. However, there are three engineered overflow structures that remain active in this vicinity (i.e. SSO 132, 134, and 135) and they transfer a large amount of sanitary sewage into the storm sewers (Table 3-3). The details of these engineered overflows and alternatives to eliminate these three engineered SSOs are provided in the Alternatives Analysis Report.

Table 3.4.A: Baseline Manhole SSO Volumes for Upper Gwynn's Run Interceptor

Manhole ID	Flow Basin	1-year (MG)	2-year (MG)	5-year (MG)	10-year (MG)	15-year (MG)	20-year (MG)
S11II1039MH	HL38	0.260	0.478	0.797	1.076	1.292	1.365
S13CC1012MH	HL35	0.0379	0.0979	0.1953	0.2791	0.3252	0.3684
S09KK1010MH	HL39	0.0082	0.0839	0.2213	0.3401	0.4126	0.4704
S13EE1005MH	HL35	0.0072	0.0412	0.0886	0.1232	0.1411	0.1553
S11GG1011MH	HL35	0.0001	0.0106	0.0376	0.0621	0.0788	0.0885
S11EE1002MH	HL36		0.0172	0.0706	0.1296	0.1654	0.1996
S07AA1012MH	HL37		0.0105	0.0411	0.0672	0.0822	0.0965
S09II1016MH	HL38		0.0099	0.0399	0.0705	0.0929	0.1006
S09CC1012MH	HL37		0.0049	0.0250	0.0433	0.0547	0.0651
S07EE1012MH	HL37		0.0023	0.0220	0.0437	0.0564	0.0678
S09MM1026MH	HL39			0.0325	0.0838	0.1189	0.1505
S11OO1021MH	HL40			0.0069	0.0130	0.0166	0.0197
S07CC1014MH	HL37			0.0039	0.0112	0.0158	0.0203
S07CC1013MH	HL37			0.0039	0.0101	0.0138	0.0174
S09KK1022MH	HL39			0.0033	0.0126	0.0195	0.0278
S11EE1005MH	HL35			0.0027	0.0099	0.0134	0.0165
S11OO1005MH	HL40			0.0023	0.0064	0.0089	0.0116
S11GG1018MH	HL38			0.0017	0.0053	0.0075	0.0096
S09CC1010MH	HL37			0.0015	0.0076	0.0117	0.0158
S09CC1020MH	HL37			0.0012	0.0024	0.0029	0.0034
S07EE1021MH	HL37			0.0007	0.0085	0.0145	0.0205
S09MM1018MH	HL40			0.0003	0.0073	0.0132	0.0194
S09OO1024MH	HL40			0.0003	0.0012	0.0016	0.0020
S13AA1028MH	HL34			0.0001	0.0059	0.0101	0.0143
S13EE1023MH	HL35				0.0035	0.0075	0.0117
S09II1005MH	HL38				0.0010	0.0037	0.0068
S13EE1002MH	HL35				0.0010	0.0030	0.0054
S07CC1002MH	HL37				0.0010	0.0019	0.0028
S13AA1006MH	HL34				0.0005	0.0020	0.0037
S09CC1015MH	HL37				0.0002	0.0004	0.0006
S09EE1004MH	HL37					0.0003	0.0019
S09II1002MH	HL38					0.0003	0.0010
S11EE1015MH	HL36					0.0001	0.0017
S13CC1018MH	HL35						0.0002
Total SSO for each return period (MG)		0.3137	0.7566	1.5993	2.4271	2.9885	3.3618

Table 3.4.B. Baseline Manhole SSO Volumes for Liberty Heights

Manhole ID	Flow Basin	1-year (MG)	2-year (MG)	5-year (MG)	10-year (MG)	15-year (MG)	20-year (MG)
S13OO_007MH	HL28			0.0771	0.1691	0.2227	0.2723
S07WW_010MH	HL32			0.0337	0.0807	0.1070	0.1300
S11WW_007MH	HL32			0.0114	0.0390	0.0572	0.0752
S09UU_013MH	HL33			0.0056	0.0178	0.0225	0.0263
S09WW_014MH	HL32			0.0025	0.0124	0.0194	0.0241
S07UU_010MH	HL33			0.0001	0.0293	0.0490	0.0678
S07WW_006MH	HL33				0.0031	0.0116	0.0184
S15KK_009MH	HL28					0.0015	0.0041
S09YY_005MH	HL32					0.0012	0.0058
S09WW_017MH	HL32					0.0007	0.0013
S09WW_004MH	HL32					0.0005	0.0029
S07WW_014MH	HL33						0.0040
S13MM_041MH	HL28						0.0010
S15KK_012MH	HL28						0.0003
S07WW_008MH	HL33						0.0002
Total SSO for each return period (MG)				0.1304	0.3514	0.4933	0.6337

Lower Gwynn's Run Interceptor

Table 3.4.C lists all flooded manholes for each design storm and Figure 3.3.C depicts the location of overflow manholes for the Lower GRI area. There are three overflow manholes for the 1-year design storm at both the upstream and downstream of the confluence point of the 30" SC812 relief and the 32" GRI near Franklin Street because these two major sewer lines tie into the existing 27" GRI along Franklin Street. According to the original design, however SC-812 was supposed to extend all the way to the HLI, but the relief sewer was shortened due to construction challenges and financial constraints. The HLSS team will develop the model with the extended portion of SC-812 to check if the SSOs near Franklin Street can be eliminated. Details will be provided in the *Alternative Analysis Report.*

West Baltimore and Eastern High Level Interceptor

Table 3.4.D lists all the overflow manholes for each design storm and Figure 3.3.D depicts the location of overflow manholes in a map for the West Baltimore and Eastern HLI area. There is one overflow manhole in the West Baltimore area for the 15-year and 20-year storms. This manhole, adjacent to the HLI, floods when the HLI significantly surcharges. In the eastern HLI area, manhole S43EE_034M is a recurring SSO manhole in front of the Baltimore City Detention Center (BCDC) which relieves over 5 MG of SSO volumes for a 5-year design storm.

There are about 1,300 ft of 8" pipe segments along the Hunter Avenue directly connected to the upstream diversion chamber of the High Level Interceptor triple-barrel siphon. One of the manholes in the section, S35CC_017MH, has experienced SSOs in the past. Many of these 8" pipe segments including the SSO manhole were added to the Baseline model recently after the capacity issues associated with the siphon were identified. Therefore, the maps in the Baseline report do not show the SSO results at S35CC_017MH. The Hunter Avenue SSO issue and simulation results corresponding to the Baseline/Future conditions will be augmented with revised graphics and text and included in the final BACA report.

Table 3.4.C Baseline Manhole SSO Volumes for Lower Gwynn's Run Interceptor

Manhole ID	Flow Basin	1-year (MG)	2-year (MG)	5-year (MG)	10-year (MG)	15-year (MG)	20-year (MG)
S15C__039MH	TSHL03	0.9351	1.2688	1.7643	2.1954	2.3864	2.6110
S15C__008MH	TSHL03	0.1736	0.2541	0.3687	0.4731	0.5374	0.5866
S15A__027MH	HL25	0.1131	0.2300	0.3896	0.5295	0.6400	0.6735
S13II_048MH	HL26		0.0734	0.1843	0.2789	0.3505	0.3778
S15EE_016MH	HL26		0.0404	0.1362	0.1982	0.2309	0.2544
S15EE_004MH	HL26		0.0013	0.0109	0.0205	0.0243	0.0313
S13EE_037MH	HL27			0.0190	0.0564	0.0732	0.0879
S13GG_042MH	HL26			0.0015	0.0041	0.0057	0.0076
S15C__011MH	TSHL03				0.0360	0.0822	0.1016
S13GG_036MH	HL27				0.0233	0.0417	0.0583
S15C__046MH	HL25				0.0007	0.0048	0.0040
S15A__040MH	HL25				0.0001	0.0021	0.0053
S21GG_009MH	HL22					0.0110	0.0419
S19II_016MH	HL23					0.0037	0.0068
S19II_031MH	HL23					0.0036	0.0119
S13II_003MH	HL27					0.0020	0.0069
S13II_031MH	HL27					0.0018	0.0032
S19KK_014MH	HL23					0.0004	0.0016
S19KK_050MH	HL23					0.0003	0.0030
S17G__019MH	TSHL03					0.0001	0.0128
S17G__036MH	TSHL03					0.0001	0.0119
S17G__029MH	TSHL03						0.0027
S21GG_035MH	HL22						0.0001
Total SSO for each return period (MG)		1.2218	1.8680	2.8745	3.8162	4.4022	4.9021

Table 3.4.D Baseline Manhole SSO Volumes for West Baltimore and Eastern High Level Interceptor

Manhole ID	Flow Basin	1-year (MG)	2-year (MG)	5-year (MG)	10-year (MG)	15-year (MG)	20-year (MG)
S37CC_034MH	HL08A	1.3529	3.0960	5.4417	7.5072	8.4736	9.4349
S43EE_049MH	TSHL01			0.0059	0.0436	0.0548	0.0529
S35CC_017MH	HL08A			0.0018	0.0185	0.0218	0.0309
S29E__016MH	HL09A					0.0160	0.0423
S43II_005MH	HL06						0.0020
Total SSO for each return period (MG)		1.3529	3.0960	5.4494	7.5693	8.5662	9.5630

3.4.4 Hydraulic Flow Restriction Under Baseline Conditions

One of the CD requirements is to identify and map all components of the wastewater collection system which restrict the flow of wastewater that cause or contribute to or are likely to cause or contribute to overflows within the collection system. InfoWorks CS has a utility function designed to assist in determining such flow restriction sections in a sewer system. InfoWorks compares the slope of the HGL at peak flow in a sewer segment to its pipe slope. A surcharged sewer with a HGL slope steeper than the pipe slope indicates that the sewer is restricting flow, i.e., a bottleneck exists in this sewer segment. If the HGL is flatter than the pipe slope, then the surcharge is not necessarily caused by a capacity limitation in that pipe and the sewer segment could be under backwater conditions caused by a downstream control.

Figure 3.4.A (Upper Gwynn's Run Interceptor), Figure 3.4.B (Liberty Heights), Figure 3.4.C (Lower Gwynn's Run Interceptor), and Figure 3.4.D (West Baltimore and Eastern High Level Interceptor) depict the results of this analysis, which shows the smallest storm event during which the sewer's capacity became restrictive and led to overflows at upstream locations.

A summary of pipe sizes and cumulative lengths identified are shown in Table 3.5

Table 3.5 Baseline Restriction Length per Pipe Size and Storm Event

Diameter (in)	3-month	1-year	2-year	5-year	10-year	15-year	20-year
8" - 14"		5,359	7,512	10,806	12,532	13,029	13,605
15" - 29"	1,600	12,131	16,114	19,492	22,748	23,772	24,445
30" - 59"	863	3,193	4,227	5,390	6,708	7,141	7,141
>= 60"	4,739	7,918	8,527	9,120	10,036	10,036	10,410
Total (ft)	7,201	28,601	36,379	44,808	52,024	53,978	55,600

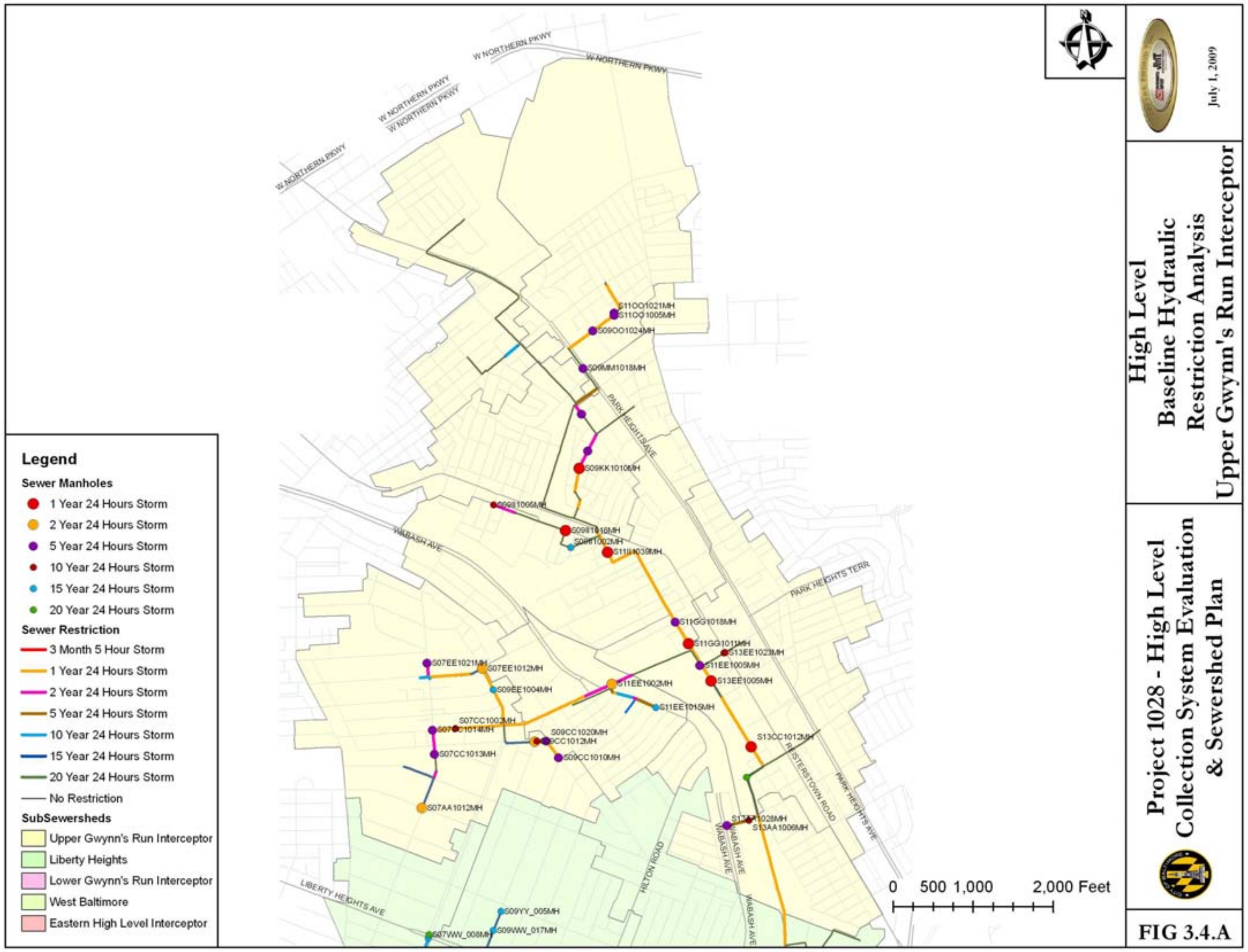


Figure 3.4.A: High Level Baseline Hydraulic Restriction Analysis Upper Gwynn's Run Interceptor

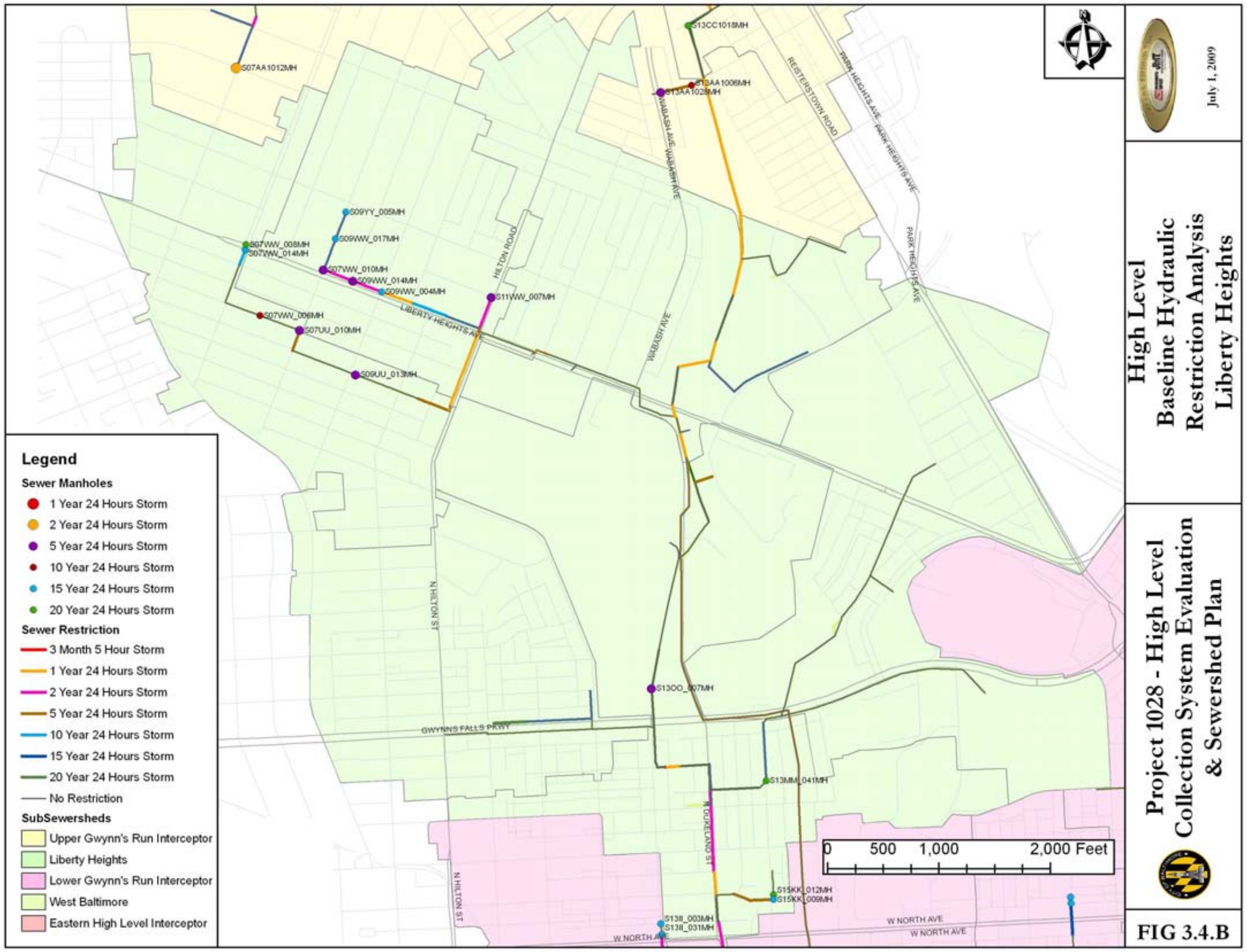


Figure 3.4.B: High Level Baseline Hydraulic Restriction Analysis Liberty Heights

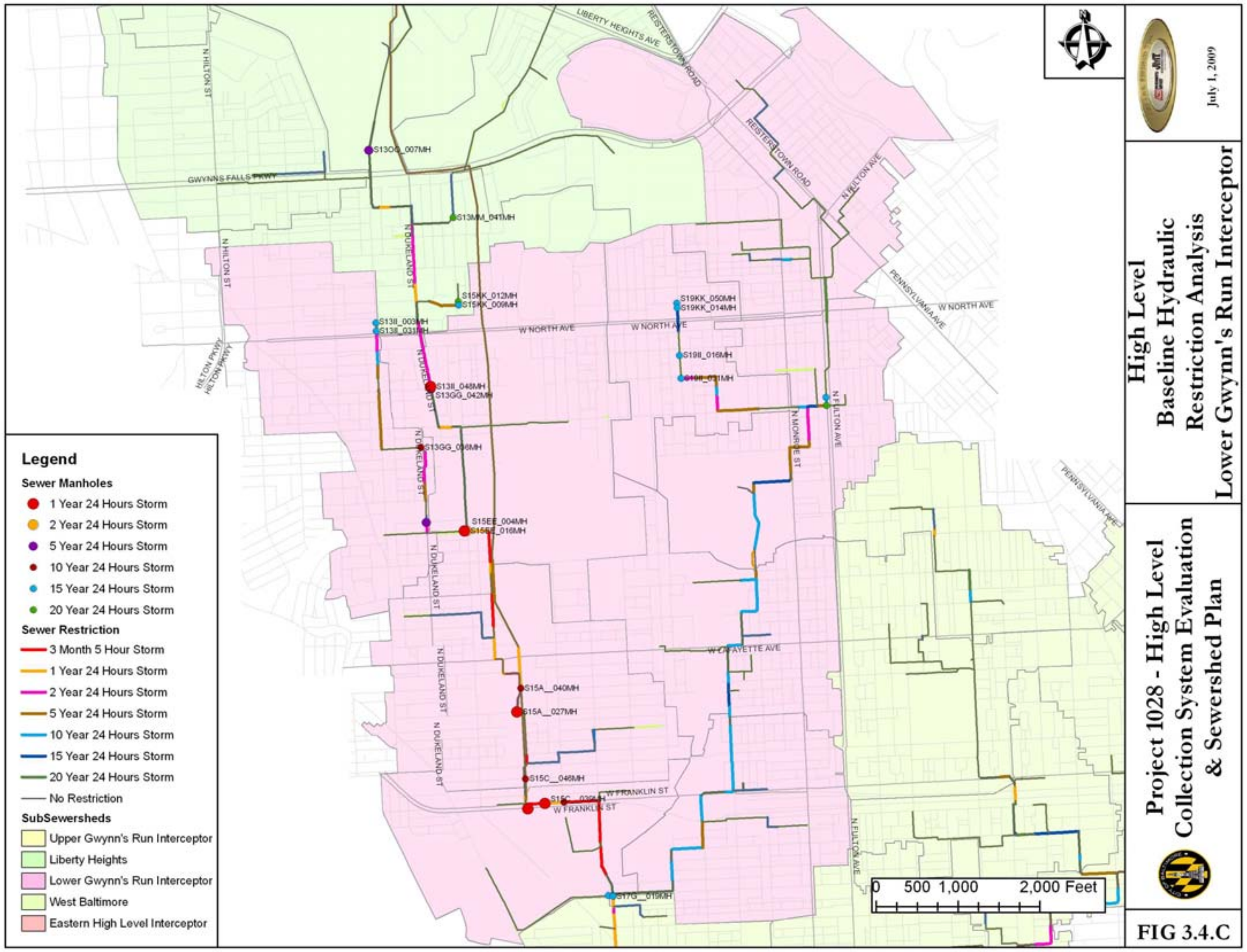


Figure 3.4.C: High Level Baseline Hydraulic Restriction Analysis Lower Gwynn's Run Interceptor

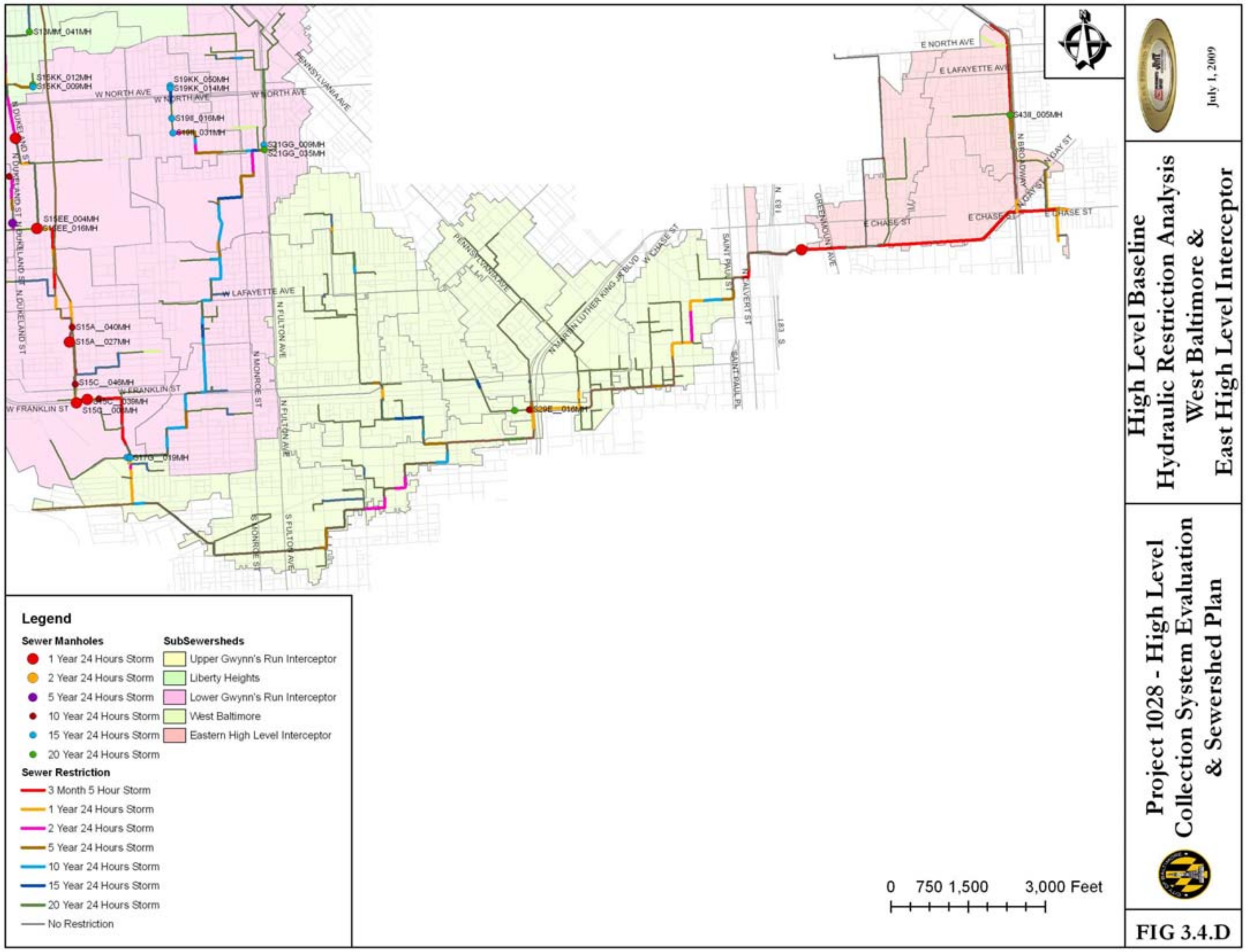


Figure 3.4.D: High Level Baseline Hydraulic Restriction Analysis East High Level Interceptor

Based on the flow restriction analysis, the HLSS team identified three major locations in the HLSS where the sewer does not have adequate capacity. These three locations are described below with each sewer profile and the maximum HGL:

Upper Gwynn's Run Interceptor

Figure 3.5.A shows the maximum HGL along the Upper GRI for the 2-year design storm condition. The HGL gradually increases from the downstream end where the SC812 relief pipe provides adequate capacity, and the HGL reaches the ground level at S13CC_1018MH. The maximum HGL is near ground elevation until manhole S09MM1006MH, where the interceptor gained a steeper slope providing an increased velocity and subsequently additional flow capacity. As shown in Table 3.4.A, S11II1039MH has the highest SSO volume because the sewer has the lowest relative depth compared to the rim elevation nearby and the HGL exceeds the manhole rim elevation for the longest duration among the Upper Gwynn's Run interceptor manholes.

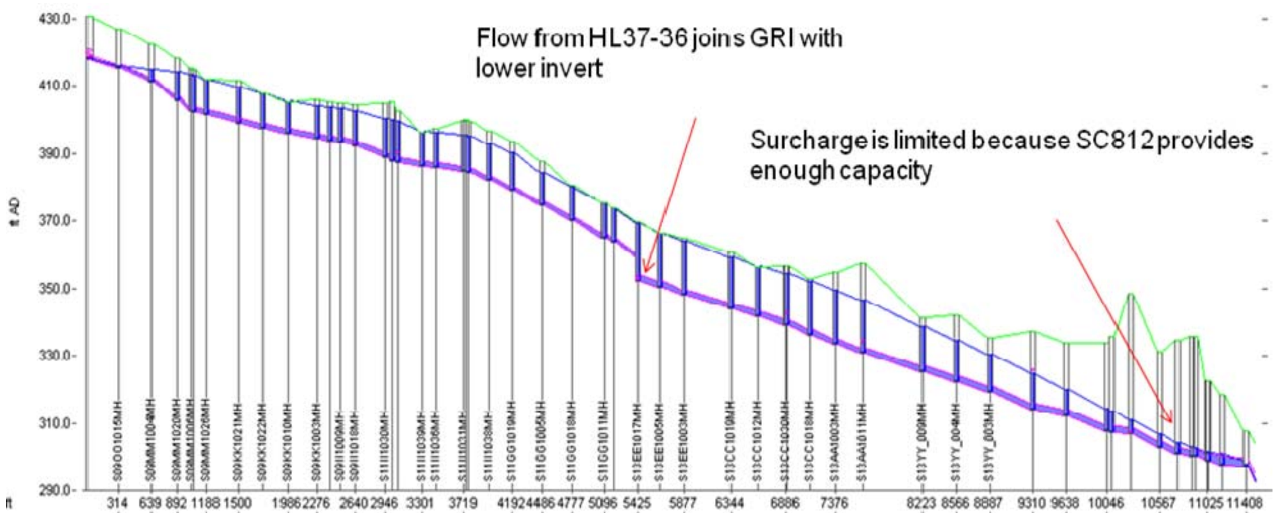


Figure 3.5.A: Maximum Hydraulic Grade Line along Upper Gwynn's Run Interceptor in 2-year Design Storm Condition.

Lower Gwynn's Run Interceptor

Figure 3.5.B shows the maximum HGL near the downstream end of GRI for the 1-year design storm condition. As mentioned in Section 3.4.3, there are three flooded manholes for the 1-year design storm near the confluence of SC812 and the existing GRI. This Figure shows that the HGL is steeper than the pipe slope for the 27" section of GRI, which implies that this 27" section has limited capacity.

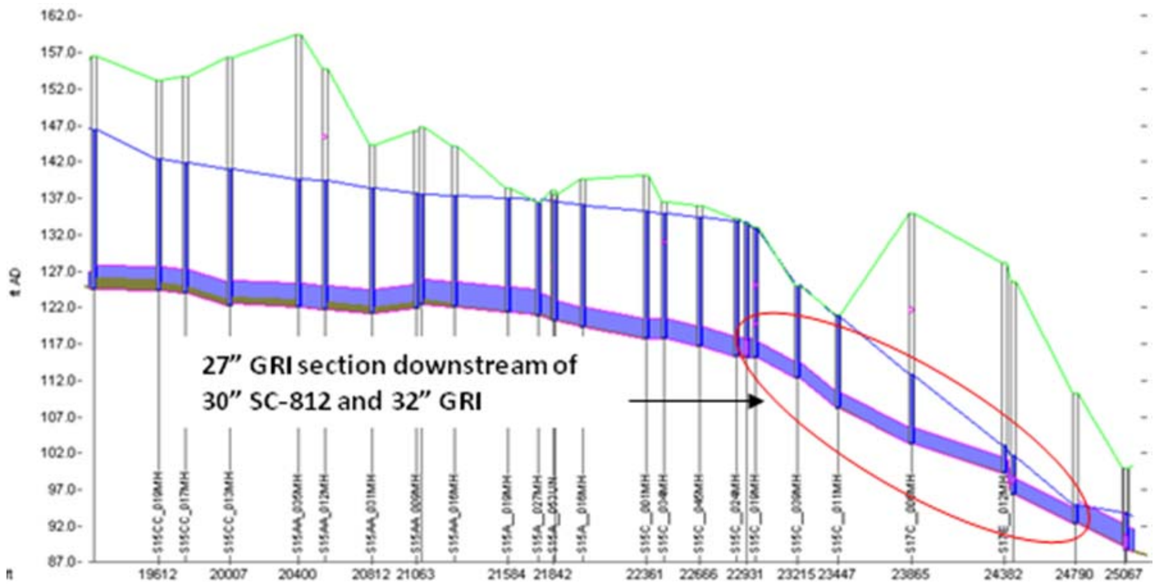


Figure 3.5.B: Maximum Hydraulic Grade Line near the Downstream End of the Gwynn's Run Interceptor in 1-year Design Storm Condition.

Eastern High Level Interceptor

Figure 3.5.C shows the maximum HGL between the triple barrel siphon and the downstream end of HLI for the 2-year design storm condition. The four major inflows from the Jones Falls and Low Level sewersheds as well as the flow from the HLSS overwhelm the Eastern High Level Interceptor which already has a diminished capacity due to heavy sediment accumulation throughout the HLI.

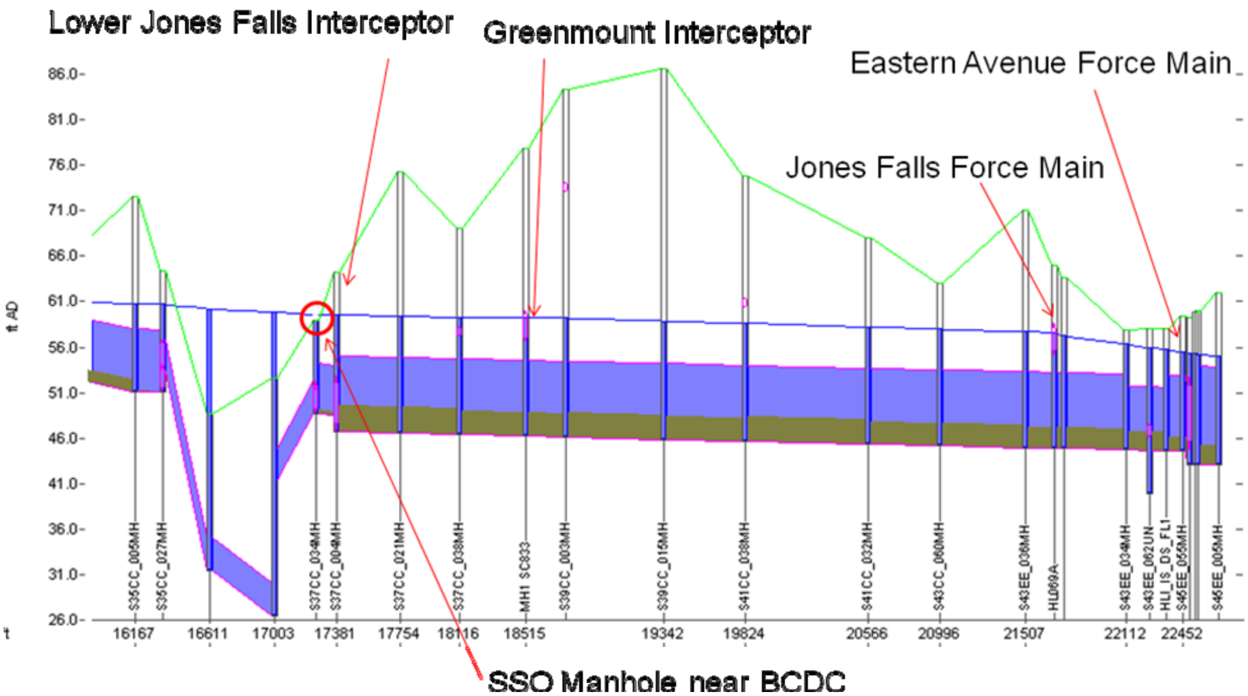


Figure 3.5.C: Maximum Hydraulic Grade Line along High Level Interceptor between Triple-barrel Inverted Siphon and the Downstream End of the High Level Interceptor in 2-year Design Storm Condition.

3.4.5 Maximum Allowable Flow Under Baseline Condition

Another requirement of the CD is to quantify the maximum flows that the capacity restricted sewer sections can handle before an overflow occurs (CD Paragraph 9.F.v.a and b). With the goal of removing SSOs from the system, the main concern is whether a component causes an overflow or not, and if so, at what flow condition does it occur. The system components identified that the lead to SSOs are discussed in Section 3.4.4.

SECTION 4

FUTURE (YEAR 2025) ANALYSIS AND CAPACITY ASSESSMENT

The CD states that the future conditions shall be based on projections for populations and sewer condition deterioration for year 2020. The City has since decided that the future projections would be based on Year 2025 to maintain consistency with the Consent Decree for the Baltimore County. Population estimates for Year 2025 that are needed to simulate future conditions are determined by the Baltimore Metropolitan Council. An ESRI shape file of the Sanitary Service Area with the future waste water discharge data was provided by the Technical Program Manager. The hydraulic impact of pipe deterioration has been represented by increasing groundwater infiltration by ten percent as stated in the BaSES manual. Detailed analysis for estimating flows based on the population projections are discussed in the City's December 2007 Report Current and Future Dry Weather Base Sanitary Flows.

4.1 DRY WEATHER CAPACITY ASSESSMENT

Similar to the dry weather capacity assessment for the baseline flow conditions, the dry weather analysis was conducted for the future Year 2025 condition. The dry weather wastewater and base infiltration rates for baseline and future conditions are listed in Table 4.1. The total difference between the future dry weather flow and baseline dry weather flow is 2.3 MGD. The ten percent increase in groundwater has been uniformly applied to each model subcatchment while the increase in wastewater has been applied proportionately to each subcatchment based on the future expected flows provided for each Sewer Service Area (SSA) from the Technical Management Team SC 1015.

Table 4.1. Dry Weather Flow Increase from Baseline to Future Conditions

	Waste Water (MGD)	Base Infiltration (MGD)	Total DWF (MGD)
Baseline	5.7	7.2	12.9
Future (2025)	7.3	7.9	15.2
Increase	1.6	0.7	2.3

As expected, the pipes that are surcharged under the baseline condition are surcharged under Year 2025 condition; however, no additional pipes appear to be surcharged applying future flow conditions to the baseline. The modeled pipes are grouped in Figure 4.1 by the percentage of filled pipe capacity at Year 2025 peak dry weather flow. No future boundary conditions have been provided by the Technical Management Team SC 1015 for dry weather analysis, as results for future conditions were not available at the time of this analysis due to the scheduling of the sewershed studies in the Back River WWTP service area. Thereby, the HLSS team has used the same boundary conditions used for the Baseline Analysis. If the currently unavailable future boundary conditions are incorporated later, the results will likely be different only for the Eastern HLI due to the influences from increased inflows into the HLI from the Jones Falls and the Low Level Sewersheds. However, the impact of the future boundary conditions on the future conditions capacity assessment is expected to be minimal. It should also be noted that the wet weather flow component remains unchanged between baseline and future conditions.

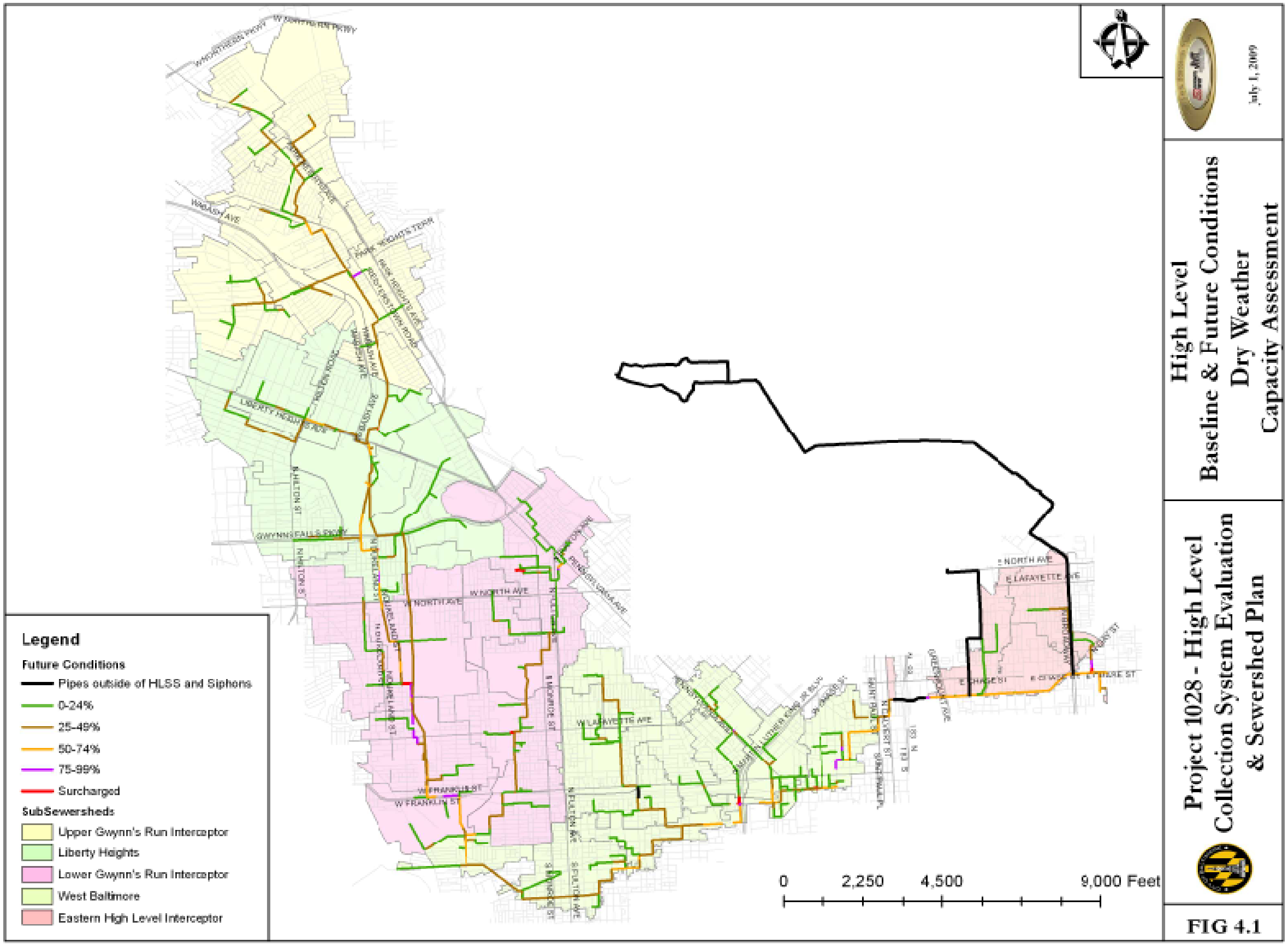


Figure 4.1: High Level Future Conditions Dry Weather Flow Capacity Assessment

4.2 WET WEATHER CAPACITY ASSESSMENT

4.2.1 Storm Events

Similar to the Baseline conditions, the same seven design storms shown in Table 3.2 are used for the wet weather capacity assessment for future conditions.

4.2.2 Boundary Conditions from Adjacent Sewersheds

Similar to the dry weather flows, the HLSS team has used the same boundary conditions used for the Baseline Analysis.

4.2.3 Return Period Analysis

The RPA capability of InfoWorks was again used to conduct this analysis. The results of the Future flooding RPA are presented for each sub-sewershed in Figure 4.2.A (Upper Gwynn's Run Interceptor), Figure 4.2.B (Liberty Heights), Figure 4.2.C (Lower Gwynn's Run Interceptor), and Figure 4.2.D (West Baltimore and East High Level Interceptor).

4.2.4 Predicted SSO Volumes

Under the DWF and 3-month storm conditions, there are no SSOs in the HLSS; but the model has predicted SSOs for all the other storm conditions analyzed. Table 4.2 shows the total SSO volumes through the manholes and each remaining engineered overflows for the 1-, 2-, 5-, 10-, 15-, and 20-year design storms. Table 4.3 shows each manhole ID and SSO volume for each corresponding storm.

Table 4.2. Future SSO Volumes

Event	Manholes (MG)	SSO132 (MG)	SSO134 (MG)	SSO135 (MG)	Garrison (MG)	Cold Spring (MG)
3-month	0.00	0.00	0.00	0.00	0.00	0.00
1-year	3.37	0.14	0.12	0.00	0.00	0.01
2-year	6.23	0.35	0.18	0.00	0.00	0.04
5-year	10.70	0.71	0.26	0.00	0.04	0.09
10-year	14.89	0.99	0.31	0.00	0.08	0.12
15-year	17.51	1.14	0.34	0.00	0.11	0.14
20-year	19.49	1.27	0.36	0.00	0.14	0.16

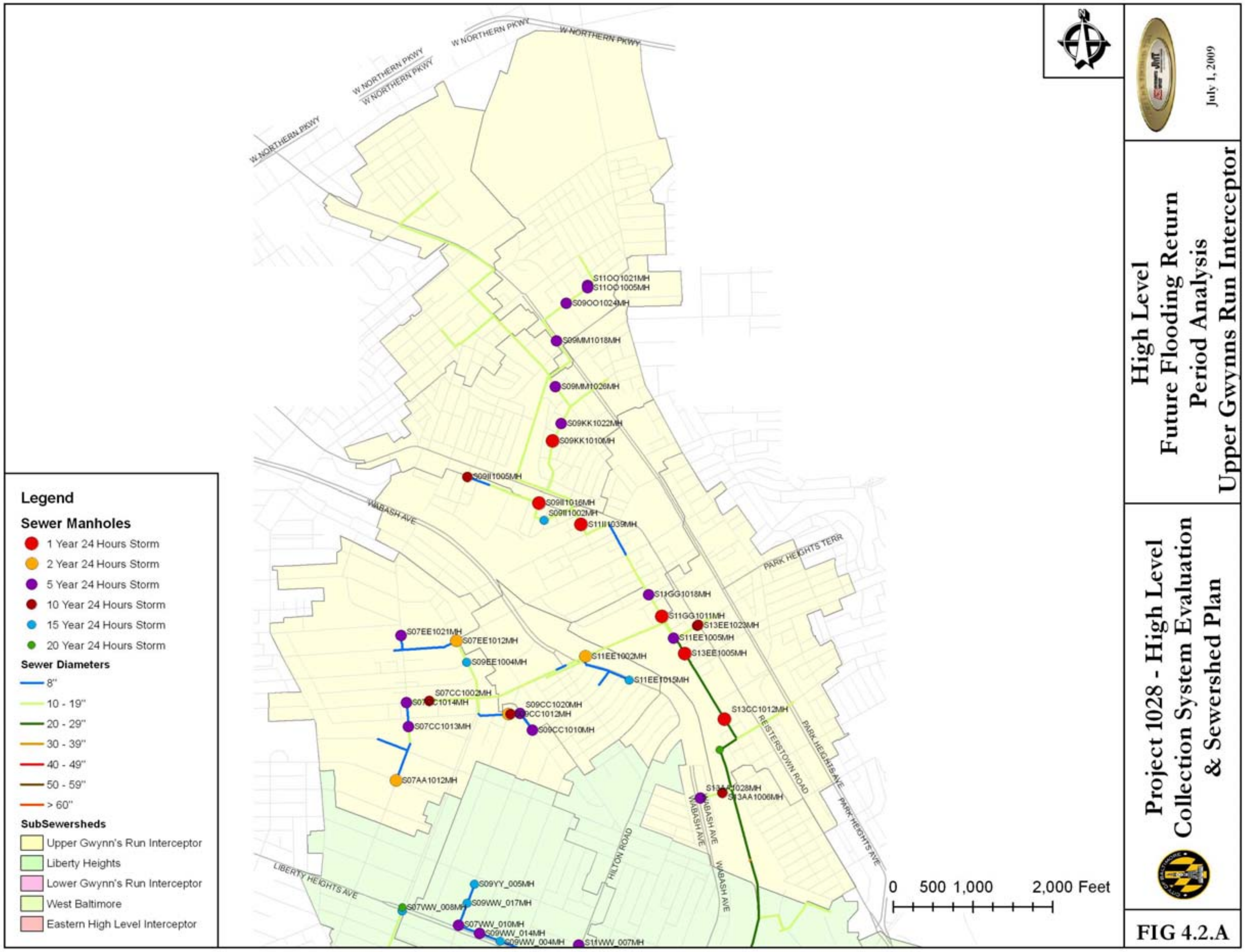


Figure 4.2.A: High Level Future Flooding Return Period Analysis Upper Gwynns Run Interceptor

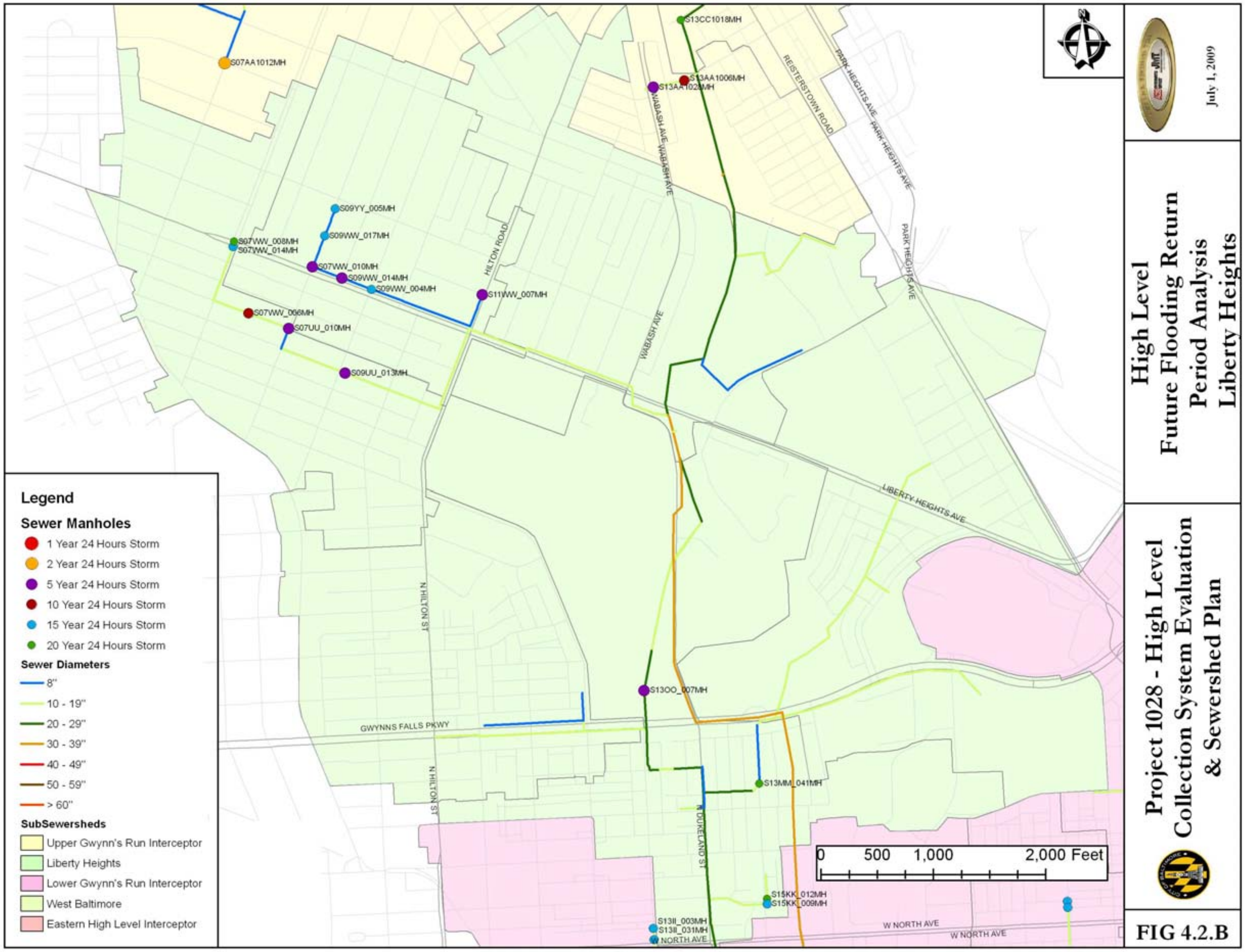


Figure 4.2.B: High Level Future Flooding Return Period Analysis Liberty Heights

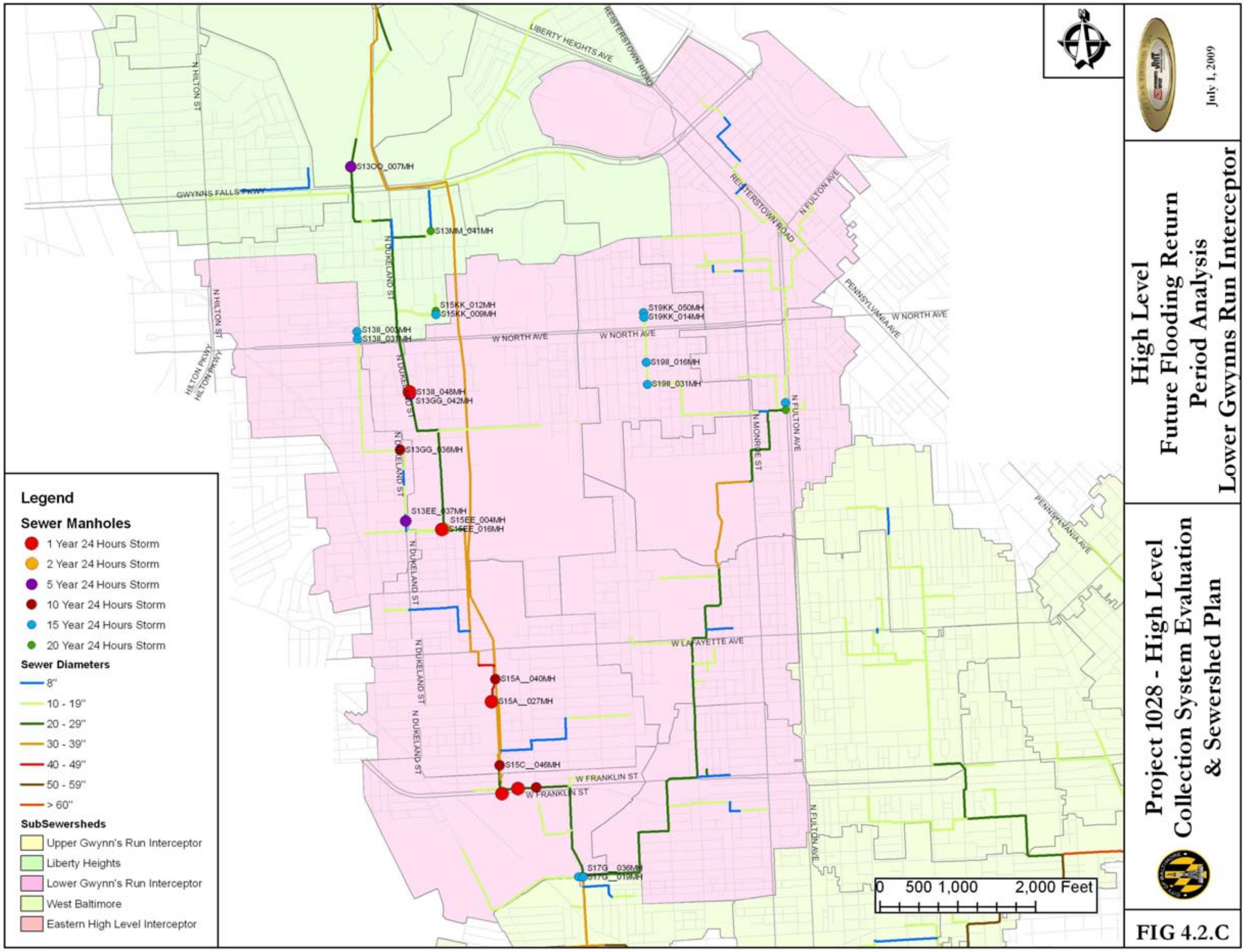


Figure 4.2.C: High Level Future Flooding Return Period Analysis Lower Gwynns Run Interceptor

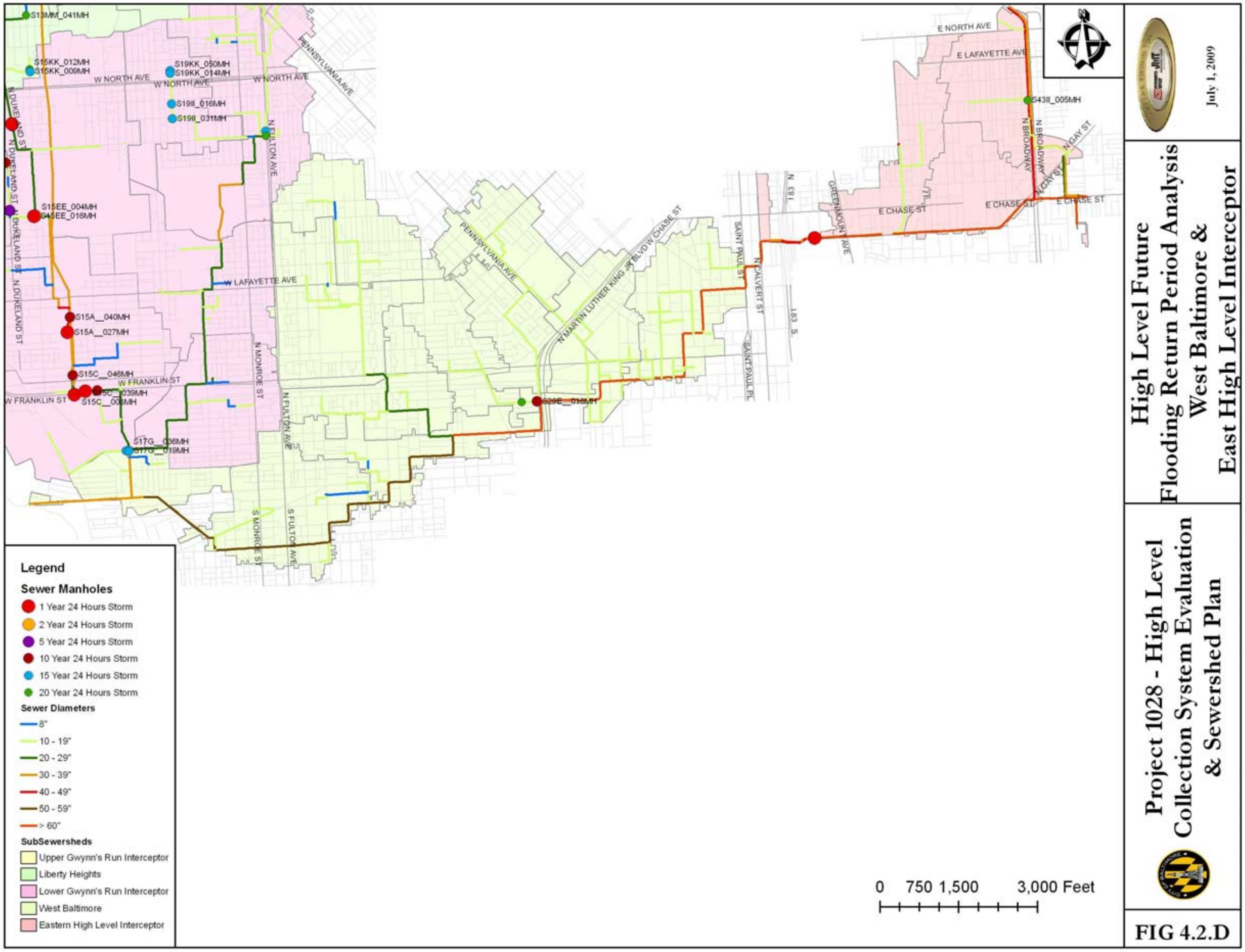


Figure 4.2.D: High Level Future Flooding Return Period Analysis West Baltimore & East High Level Interceptor

Table 4.3 Future (2025) Manhole SSO Volumes

Manhole ID	Flow Basin	Sub-Sewershed	1-year (MG)	2-year (MG)	5-year (MG)	10-year (MG)	15-year (MG)	20-year (MG)
S11II1039MH	HL38	Upper GRI	0.2780	0.4979	0.8216	1.1046	1.4453	1.4867
S13CC1012MH	HL35	Upper GRI	0.0405	0.1001	0.1977	0.2820	0.3271	0.3722
S09KK1010MH	HL39	Upper GRI	0.0105	0.0889	0.2276	0.3481	0.4240	0.4865
S13EE1005MH	HL35	Upper GRI	0.0089	0.0429	0.0904	0.1251	0.1455	0.1589
S11GG1011MH	HL35	Upper GRI	0.0002	0.0114	0.0386	0.0635	0.0863	0.0937
S09II1016MH	HL38	Upper GRI	0.0001	0.0107	0.0412	0.0721	0.1008	0.1101
S11EE1002MH	HL36	Upper GRI		0.0200	0.0757	0.1365	0.1728	0.2081
S07AA1012MH	HL37	Upper GRI		0.0108	0.0414	0.0677	0.0823	0.0971
S09CC1012MH	HL37	Upper GRI		0.0051	0.0254	0.0439	0.0567	0.0659
S07EE1012MH	HL37	Upper GRI		0.0025	0.0224	0.0441	0.0568	0.0684
S09MM1026MH	HL39	Upper GRI			0.0337	0.0856	0.1226	0.1553
S11OO1021MH	HL40	Upper GRI			0.0070	0.0132	0.0173	0.0201
S07CC1014MH	HL37	Upper GRI			0.0041	0.0114	0.0161	0.0205
S07CC1013MH	HL37	Upper GRI			0.0039	0.0102	0.0143	0.0175
S09KK1022MH	HL39	Upper GRI			0.0036	0.0132	0.0209	0.0287
S11EE1005MH	HL35	Upper GRI			0.0029	0.0101	0.0149	0.0167
S11OO1005MH	HL40	Upper GRI			0.0024	0.0065	0.0087	0.0117
S11GG1018MH	HL38	Upper GRI			0.0018	0.0054	0.0090	0.0097
S09CC1010MH	HL37	Upper GRI			0.0015	0.0077	0.0118	0.0159
S09CC1020MH	HL37	Upper GRI			0.0012	0.0024	0.0032	0.0034
S07EE1021MH	HL37	Upper GRI			0.0008	0.0086	0.0146	0.0206
S09MM1018MH	HL40	Upper GRI			0.0004	0.0075	0.0135	0.0197
S09OO1024MH	HL40	Upper GRI			0.0003	0.0012	0.0017	0.002
S13AA1028MH	HL34	Upper GRI			0.0002	0.0060	0.0102	0.0144
S13EE1023MH	HL35	Upper GRI				0.0036	0.0077	0.0118
S09II1005MH	HL38	Upper GRI				0.0011	0.0039	0.0071
S13EE1002MH	HL35	Upper GRI				0.0011	0.0031	0.0055
S07CC1002MH	HL37	Upper GRI				0.0011	0.0020	0.0029
S13AA1006MH	HL34	Upper GRI				0.0006	0.0021	0.0038
S09CC1015MH	HL37	Upper GRI				0.0003	0.0003	0.0006
S09II1002MH	HL38	Upper GRI					0.0013	0.0010
S11EE1015MH	HL36	Upper GRI					0.0003	0.0022
S09EE1004MH	HL37	Upper GRI					0.0003	0.0020

Table 4.3 Future (2025) Manhole SSO Volumes

Manhole ID	Flow Basin	Sub-Sewershed	1-year (MG)	2-year (MG)	5-year (MG)	10-year (MG)	15-year (MG)	20-year (MG)
S13CC1018MH	HL35	Upper GRI						0.0003
Upper Gwynn's Run Interceptor Subtotal			0.3382	0.7903	1.6458	2.4844	3.1974	3.5410
S13OO_007MH	HL28	Liberty Heights			0.0802	0.1731	0.2274	0.2765
S07WW_010MH	HL32	Liberty Heights			0.0339	0.0810	0.1063	0.1297
S11WW_007MH	HL32	Liberty Heights			0.0114	0.0392	0.0571	0.0754
S09UU_013MH	HL33	Liberty Heights			0.0061	0.0181	0.0231	0.0268
S09WW_014MH	HL32	Liberty Heights			0.0026	0.0124	0.0203	0.0249
S07UU_010MH	HL33	Liberty Heights			0.0003	0.0303	0.0497	0.0690
S07WW_006MH	HL33	Liberty Heights				0.0034	0.0124	0.0189
S15KK_009MH	HL28	Liberty Heights					0.0017	0.0042
S09YY_005MH	HL32	Liberty Heights					0.0012	0.0059
S09WW_017MH	HL32	Liberty Heights					0.0010	0.0013
S09WW_004MH	HL32	Liberty Heights					0.0005	0.0030
S07WW_014MH	HL33	Liberty Heights					0.0001	0.0042
S13MM_041MH	HL28	Liberty Heights						0.0015
S15KK_012MH	HL28	Liberty Heights						0.0004
S07WW_008MH	HL33	Liberty Heights						0.0003
Liberty Heights Subtotal					0.1345	0.3575	0.5008	0.6420
S15C__039MH	TSHL03	Lower GRI	0.9962	1.3332	1.8400	2.2712	2.4495	2.6765
S15C__008MH	TSHL03	Lower GRI	0.1821	0.2660	0.3830	0.4910	0.5542	0.6113
S15A__027MH	HL25	Lower GRI	0.1255	0.2429	0.4055	0.5503	0.6839	0.7403
S13II_048MH	HL26	Lower GRI	0.0009	0.0795	0.1905	0.2862	0.4393	0.4073
S15EE_016MH	HL26	Lower GRI	0.0004	0.0437	0.1403	0.2026	0.2349	0.2608
S15EE_004MH	HL26	Lower GRI		0.0016	0.0117	0.0215	0.0251	0.0309
S13EE_037MH	HL27	Lower GRI			0.0204	0.0574	0.0745	0.0888
S13GG_042MH	HL26	Lower GRI			0.0016	0.0043	0.0031	0.0076
S15C__011MH	TSHL03	Lower GRI				0.0459	0.0979	0.1184
S13GG_036MH	HL27	Lower GRI				0.0244	0.0431	0.0593
S15C__046MH	HL25	Lower GRI				0.0009	0.0218	0.0094
S15A__040MH	HL25	Lower GRI				0.0002	0.0041	0.0056
S21GG_009MH	HL22	Lower GRI					0.0144	0.0464
S17G__019MH	TSHL03	Lower GRI					0.0052	0.0211
S17G__036MH	TSHL03	Lower GRI					0.0051	0.0195

Table 4.3 Future (2025) Manhole SSO Volumes

Manhole ID	Flow Basin	Sub-Sewershed	1-year (MG)	2-year (MG)	5-year (MG)	10-year (MG)	15-year (MG)	20-year (MG)
S19II_031MH	HL23	Lower GRI					0.0042	0.0126
S19II_016MH	HL23	Lower GRI					0.0039	0.007
S13II_003MH	HL27	Lower GRI					0.0022	0.0072
S13II_031MH	HL27	Lower GRI					0.0019	0.0033
S17G__029MH	TSHL03	Lower GRI					0.0008	0.0045
S19KK_014MH	HL23	Lower GRI					0.0005	0.0017
S19KK_050MH	HL23	Lower GRI					0.0004	0.0032
S21GG_035MH	HL22	Lower GRI						0.0003
Lower Gwynn's Run Interceptor Subtotal			1.3051	1.9669	2.9930	3.9559	4.6700	5.1430
S37CC_034MH	HL08A	Eastern HLI	1.7276	3.4714	5.9195	8.0396	9.0445	10.0428
S43EE_049MH	TSHL01	Eastern HLI			0.0064	0.0454	0.0672	0.0616
S35CC_017MH	HL08A	Eastern HLI			0.0064	0.0220	0.0239	0.0310
S43II_005MH	HL06	Eastern HLI						0.0024
S35AA_023MH	HL08A	Eastern HLI						0.0002
Eastern High Level Interceptor Subtotal			1.7276	3.4714	5.9323	8.1070	9.1356	10.1380
S29E__016MH	HL09A	West Baltimore				0.0042	0.0265	0.0553
S29E__009MH	HL09A	West Baltimore						0.0002
West Baltimore Subtotal						0.0042	0.0265	0.0555

4.2.5 Future Hydraulic Flow Restriction

Similar to the Flow Restriction analysis conducted for the Baseline Conditions, the analysis performed for future conditions revealed several pipe segments with flow restrictions. Figure 4.3.A (Upper Gwynn's Run Interceptor), Figure 4.3.B (Liberty Heights), Figure 4.3.C (Lower Gwynn's Run Interceptor), and Figure 4.3.D (West Baltimore and Eastern High Level Interceptor) depict the results of this analysis, showing the smallest storm event that resulted in an upstream overflow. As shown in these maps, no major difference was observed between the Baseline and Future conditions.

4.2.6 Future Maximum Allowable Flow Before an Overflow

One of the requirements of the Consent Decree is to identify system components that restrict flow and quantify the maximum flows that the identified components can handle before an overflow occurs (CD Paragraph 9.F.v.a and b). With the goal of removing SSOs from the system, the main concern is whether a component causes an overflow or not, and if so, when does it occur.

System behaviors in the Baseline and Future conditions during wet weather have no significant difference. Thus the system components identified that lead to SSOs and their level of service provided (storm return period that causes an overflow) for the Baseline condition, discussed in Section 3.4.3, can be applied to the Future condition also.

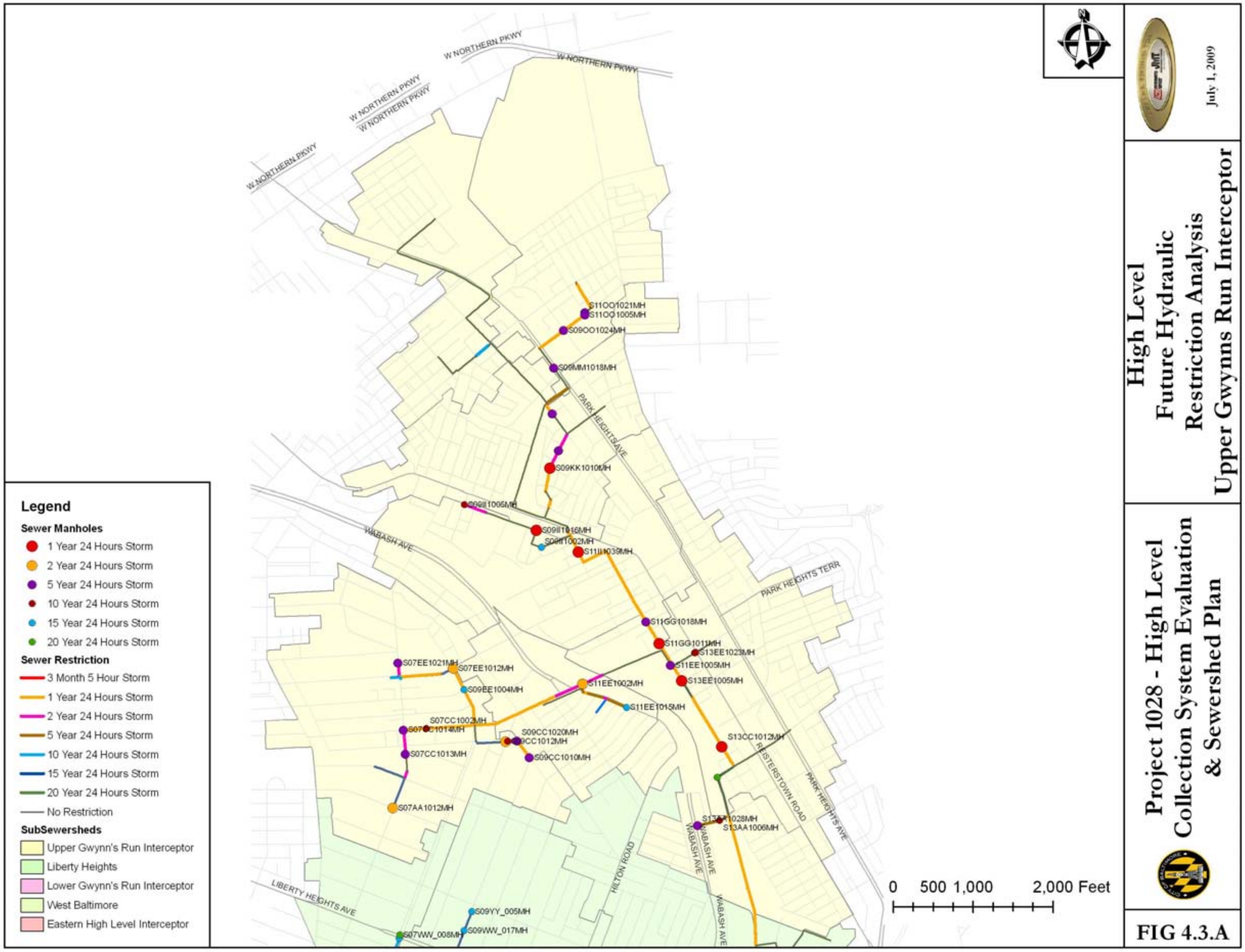


Figure 4.3.A: High Level Future Hydraulic Restriction Analysis Upper Gwynns Run Interceptor

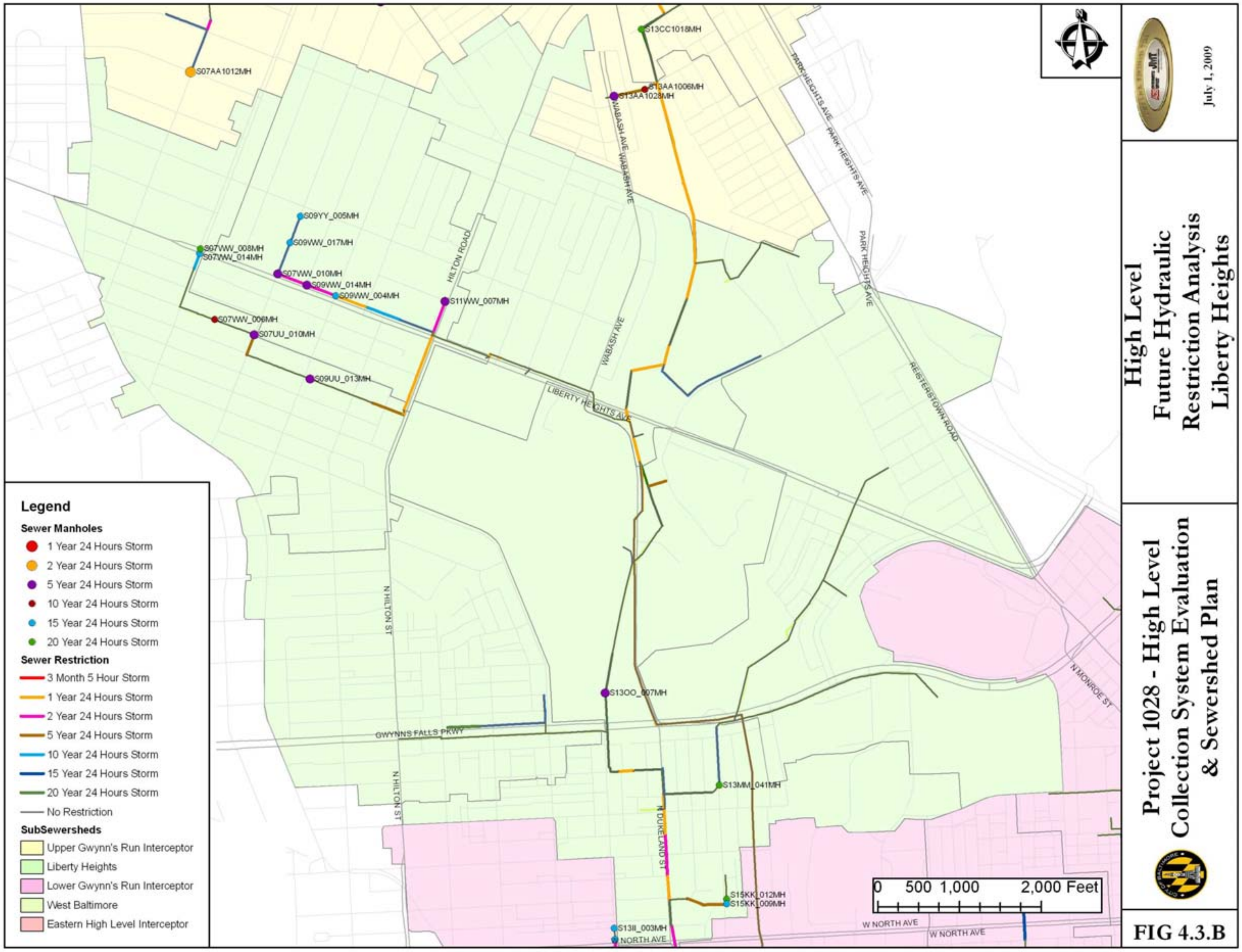


Figure 4.3.B: High Level Future Hydraulic Restriction Analysis Liberty Heights

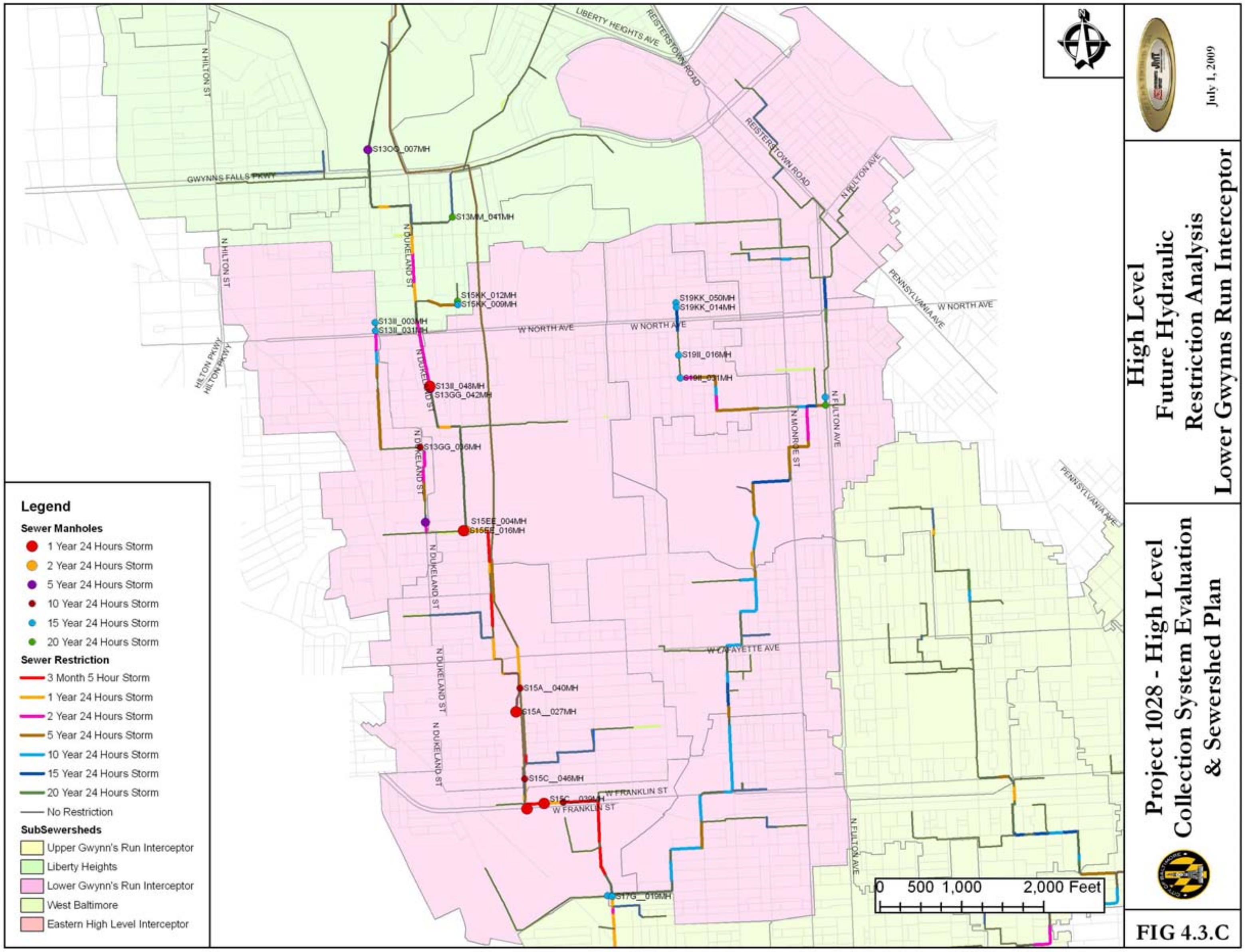


Figure 4.3.C: High Level Future Hydraulic Restriction Analysis Lower Gwynns Run Interceptor

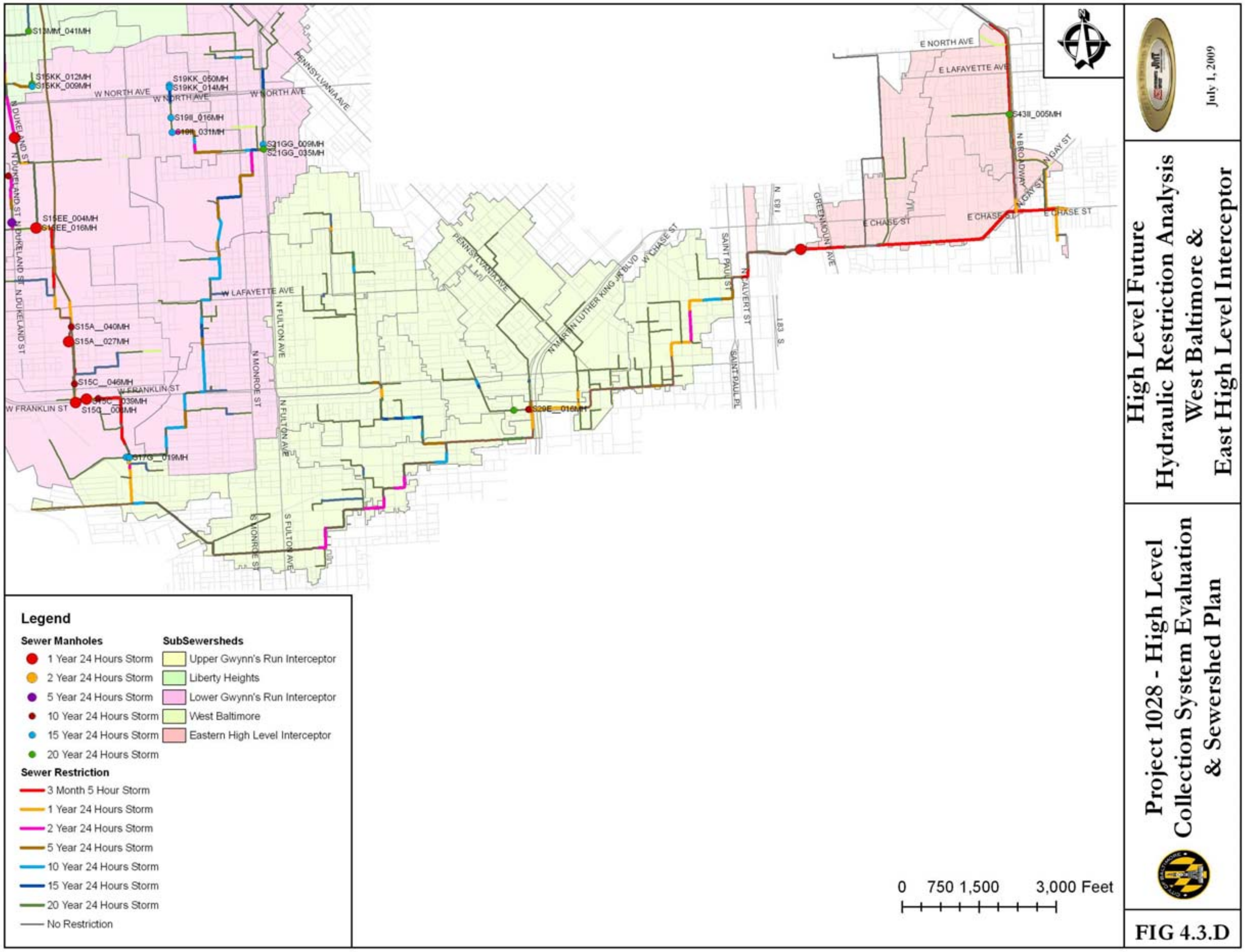


Figure 4.3.D: High level Future Hydraulic Restriction Analysis West Baltimore & East High Level Interceptor